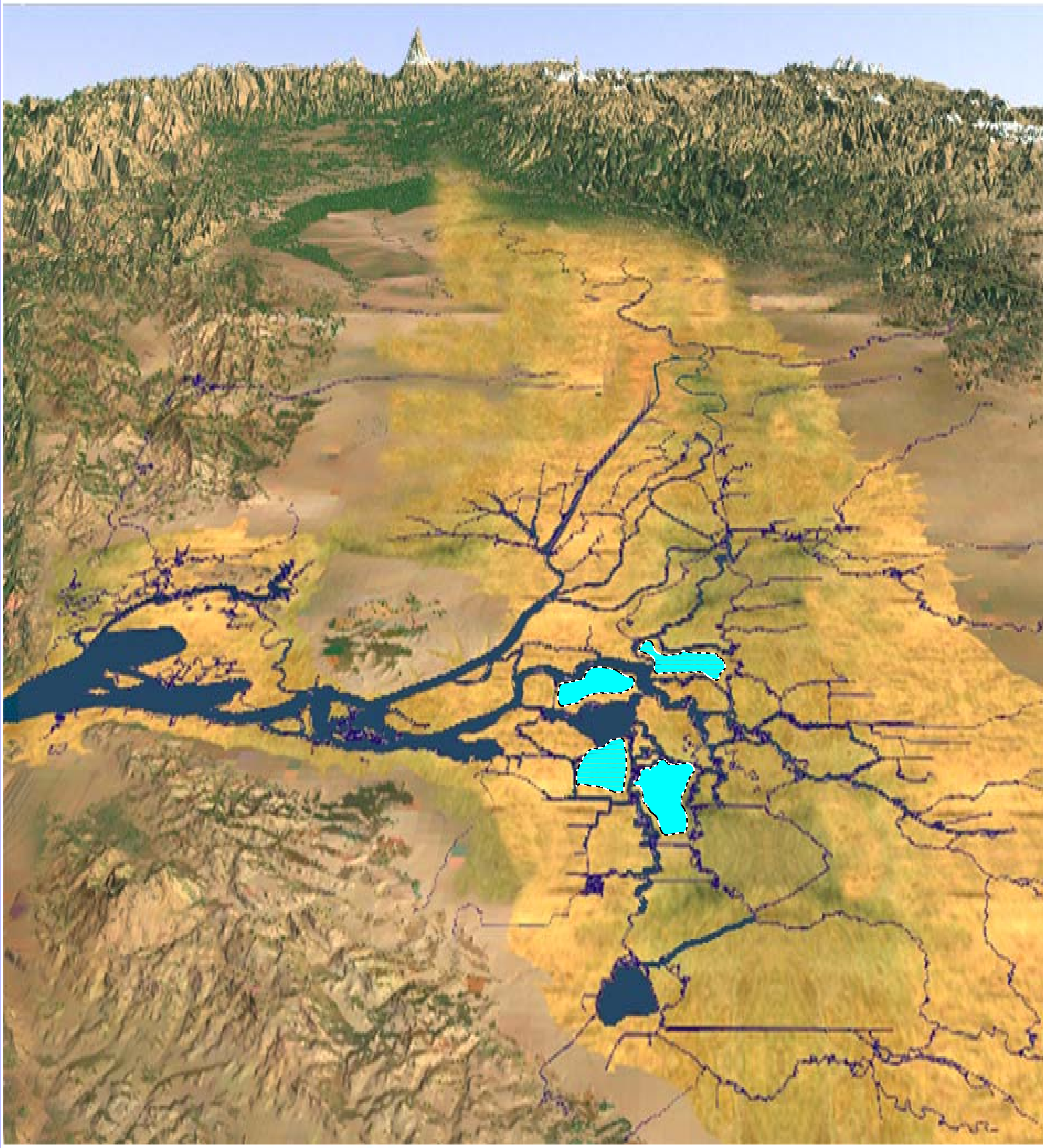
INTEGRATED STORAGE INVESTIGATIONS

IN-DELTA STORAGE PROGRAM
DRAFT REPORT ON ENGINEERING INVESTIGATIONS



CALFED BAY-DELTA PROGRAM

May 2002

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ACRONYMS

CALFED	CALFED Bay Delta Program
CCFB	Clifton Court Forebay
CDEC	California Data Exchange Center
CFS	Cubic Feet per Second
CM/S	Centimeter per second
CPT	Cone Penetrometer Test
DSOD	Division of Safety of Dams
DW	Delta Wetlands
DWR	Department of Water Resources
EIR/EIS	Environmental Impact Report/Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FT	Feet
HLA	Harding Lawson Associates
IEP	Interagency Ecological Program
K	Hydraulic Conductivity (centimeters per second)
MSL	Mean sea level
MW	Megawatt
PG&E	Pacific Gas & Electricity
PSF	Pound per square foot
PSHA	Probabilistic Seismic Hazard Analysis
ROD	Record of Decision
SPT	Standard Penetration Test
SWRCB	State Water Resources Control Board
TAF	Thousand Acre Feet
URS	URS Greiner Woodward Clyde
USACE	United State Army Corps of Engineers
RECLAMATION	United States Bureau of Reclamation

CHAPTER 1 – EXECUTIVE SUMMARY

1.1 General

The CALFED Bay-Delta Program was established in 1995 to develop a long-term comprehensive plan to restore ecological health and improve water management for beneficial uses of the Bay-Delta. The Integrated Storage Investigations Program was initiated by CALFED in 1999 to support efforts toward meeting the goals defined under CALFED's comprehensive water management strategy. The June 2000 plan titled *"California's Water Future: A Framework for Action"* stated that a CALFED's primary goal was to improve the reliability of California's water supply. Developing new storage was an important component of the overall CALFED strategy to meet the competing environmental and other water supply needs.

The CALFED 2000 Record of Decision (ROD) identifies In-Delta Storage as one of five surface storage projects (Shasta enlargement, Los Vaqueros, In-Delta, Sites and San Joaquin Storage) to be pursued by a project-specific study early in Stage 1 of the Bay-Delta Program implementation. The purpose of new storage in the Delta is to increase operational flexibility for the Central Valley Project (CVP) and the State Water Project (SWP) and to provide ecosystem benefits in the Delta. The ROD includes an option to explore the lease or purchase of the Delta Wetlands Project (DW Project), a private proposal by Delta Wetlands Properties or to initiate a new project in the event that DW Project proves cost prohibitive or infeasible.

The ROD established the following decision points for the In-Delta Storage Program:

- Proceed to seek federal authorization for a feasibility study by October 2000.
- Select a project alternative and initiate negotiation with Delta Wetlands' owners or other appropriate landowners for acquisition of necessary property by December 2001.
- Develop a project plan that addresses local concerns about effects on neighboring lands, and complete any additional needed environmental documentation by July 2002.
- Complete environmental review and documentation, obtain necessary authorization and funding, and begin construction by the end of 2002.

A joint U.S. Bureau of Reclamation (Reclamation) and California Department of Water Resources (DWR) reconnaissance-level study of the DW Project and other potential alternatives, completed in September 2000, concluded that the In-Delta Storage Project would meet goals of operational flexibility and provide other beneficial uses in the Delta.

The participating agencies initiated a specific project study of the In-Delta Storage Project in January 2001 including investigations of operations, water quality, engineering, environmental impacts, as well as economic, policy and legal evaluations. The engineering investigation information presented in this report was produced to assess the technical feasibility of the DW Project, as proposed, for public ownership, and to recommend alternative modifications or improvements to it, if deemed necessary. Two additional reports, "*In-Delta Storage Program Risk Analyses*", prepared by URS Corporation (URS) and "*Draft Report, Evaluation of Delta Wetlands Fish Screens, Siphons, and Pumping Structures Evaluations*", prepared jointly by URS and CH2M HILL as part of this engineering investigation, are available separately. Conclusions and recommendations are presented in the Executive Summary.

1.2 In-Delta Storage Alternatives

1.2.1 Delta Wetlands Project

Delta Wetlands Properties, a private company, proposed a water storage project in the Sacramento-San Joaquin Delta involving conversion of Webb Tract and Bacon Island into storage reservoirs, termed "reservoir islands", and conversion of Bouldin Island and Holland Tract into "habitat islands". The DW Project proposed to divert and store surplus water in reservoir islands and make seasonal diversions to habitat islands for wetland and wildlife enhancement as environmental mitigation (Figure 1). The total storage capacity of the two reservoir islands was estimated at approximately 216 thousand acre-feet (TAF) with a designated water surface elevation of 4 feet above mean sea level (MSL). Delta Wetlands Properties submitted a Draft EIR in 1995 to the State Water Resources Control Board pursuant to water rights filings. The EIR was revised in July 2000 and permits were issued in February 2001. The maximum permitted diversion to the reservoir islands and habitat islands was set at 9,000 cfs and the maximum permitted release was set at 6,000 cfs. The U.S. Army Corps of Engineers (USACE) approval of the EIS to meet the 404 permit requirements was pending at the time this report was completed.

Details of the embankment sections proposed in the DW Project were presented in the February 15, 1989, Geotechnical Investigation Report by Harding Lawson Associates (HLA). URS reviewed HLA's geotechnical information in May 2000. The results of URS' review were presented in the State Water Resources Control Board July 2000 EIR. The final 2001 EIS specified a final embankment section with a crest width of 22 feet, a crest elevation of 9 feet above MSL and a 5:1 constant reservoir-side slope. As an alternative, a variable slope section with a 3:1 upper section slope (top of crest to elevation -3) and 10:1 lower section slope was considered.

Each of the proposed DW Project reservoir islands would have new siphon and pump stations for the diversion and release of water, respectively, located as shown in Figures 2 and 3. For the diversion of water, each reservoir island would have two new siphon stations consisting of 16 siphons each (64 siphons in total). The siphons would be spaced 40 feet apart and would be equipped with booster pumps, flow meters, and barrel type fish screens. The 57 existing siphons located on the perimeter of the reservoir and habitat islands would be retrofitted with barrel type fish screens similar to those proposed for the new siphon stations. For release or discharge of water, a discharge station is proposed at each island, with the station at Webb Tract having 32 pumps and the one at Bacon Island having 40 pumps. Diversion of water onto habitat islands will be accomplished through modified existing siphons. There will be 14 siphons on Bouldin Island and 8 siphons on Holland Tract, all retrofitted with fish screens similar to those installed at the reservoir islands.

To prevent seepage of stored water to the neighboring islands, 773 pump-equipped interceptor wells would be installed around the perimeter of the reservoir islands to intercept and pump water back to the reservoirs.

1.2.2 Re-engineered Delta Wetlands Project

As part of the In-Delta Storage evaluation and for improving the storage operations, Reclamation/DWR considered a Re-engineered Alternative. This alternative included the same reservoir islands and habitat islands as the DW Project but changed the design of the embankments and consolidated the 99 diversion siphons and 72 discharge pumps into four Integrated Facilities with each island containing two such facilities, located as shown in Figure 4 and 5.

1.2.3 Victoria Island with Connection to Clifton Court

To further improve the DW Project operational flexibility, two additional storage alternatives were considered. The first of these proposed replacing Webb Tract with Victoria Island and provided for a direct connection of Victoria Island to Clifton Court. In this case, Bacon Island and Victoria Island would be the reservoir islands and diversion and release of water would be realized through the use of two Integrated Facilities on each island. In addition, water would siphon directly from Victoria Island to Clifton Court by gravity or would flow through a new siphon-and-pumping-combination conveyance facility. The other alternative proposed using Webb Tract and Victoria Island as the reservoir islands. In this alternative also, diversion and release of water would be realized through the use of two Integrated Facilities on each island, and, water would be siphoned directly from Victoria Island to Clifton Court by gravity or flow through a new siphon-and-pumping-combination conveyance facility. The Integrated Facilities and Conveyance Facilities would be located as shown in Figures 4 through 7.

1.3 Scope Of Work

The purpose of this engineering investigation was to evaluate the adequacy of the proposed DW Project design and provide recommendations for design modifications as applicable, for public ownership of the project, in the event federal and state agencies decide to buy or lease it. The scope of work included detailed engineering investigations conducted jointly by Reclamation and DWR to evaluate the technical feasibility of the DW Project and alternative modifications for improvements to it. In the evaluation, Reclamation/DWR used the information contained in the EIR/EIS and in other reports pertaining to the DW Project along with new information, such as mapping and geotechnical explorations, performed by Reclamation/DWR during summer 2001.

The Reclamation/DWR evaluation focused on the following areas:

- Project hydrology
- Field geotechnical and mapping investigations
- DW Project proposed embankment design
- Technical viability of the DW Project proposed fish screens, siphons and pump stations
- Risk assessment for potential failure of the DW Project because of operational, seismic, and flood events including seepage to adjacent islands
- Reclamation/DWR proposed design for embankment and structures in relation to improvements in design for a re-engineered project or a reconfigured project
- Project material quantities and costs

1.4 Conclusions and Recommendations

An evaluation of the DW Project was conducted consistent with the CALFED ROD directive. This evaluation was based on engineering design reviews, geotechnical investigations, risk analyses and a Board of Consultants' evaluation of the proposed embankments, fish screens, siphons and pumping stations. The evaluation concluded that the proposed embankments and inlet/outlet structural designs for Webb Tract and Bacon Island did not meet the Reclamation/DWR design requirements for public ownership of the project by these two agencies.

1.4.1 Delta Wetlands Embankment Design

1.4.1.1 Findings and Conclusions

1.4.1.1.1 Slope Stability

- **End-of-Construction:** The probability of embankment failure with release of water from the adjacent river or slough into the reservoir area was determined to be significant (greater than 50 percent), if construction proceeded too rapidly or without staging. It was assumed construction of the embankments would be carried out over a minimum of 5 years to reduce the probability of failure.
- **Long-term Condition at Steady-State Seepage:** The long-term (steady-state) factors of safety met or exceeded all design criteria for sliding toward the island. Factors of safety for sliding toward the river/slough did not meet any of the design criteria. This potential problem primarily existed where the

channel was deep and resulted in marginal to unacceptable risk of failure. The potential consequences include environmental damage, damage to floating structures, damage to adjacent levees and would require expensive clean-up measures (dredging, for example).

- **Sudden Drawdown:** The study also recognized that some of the reservoir embankment sections had inadequate factors of safety for the sudden drawdown condition, and revisions to the proposed configuration would be required during final design in these areas.

1.4.1.1.2 Seepage

Seepage analyses indicated that water levels under adjacent islands would rise because of the DW Project. The potential for seepage-induced piping and erosion could be high if high water heads are allowed to build up, without seepage control measures. The DW Project provides for the construction of interceptor wells to control adverse seepage conditions. This system will have high operation and maintenance costs that need to be accounted for in the overall cost of the DW Project. The interceptor well system is prone to failures because of geotechnical conditions and because of local power interruptions or major power failures. For example, power loss or grid failures may last from days to weeks, or even months, in the event of a major earthquake. While backup (e.g., diesel operated pumps) is contemplated for the well system, local or distant earthquakes could cause extended power failures, or even prevent or limit access to the backup pumps for a significant duration of time. Interruptions in the operation of the system could cause localized flooding of adjacent islands.

1.4.1.1.3 Seismic

The seismic-induced deformations predicted by URS (2000) would result in severe cracking and possible failure from erosion through cracks or an overtopping failure because of slumping and loss of freeboard. The DW Project concluded that some liquefaction and failure of the embankments could occur, but the potential for this would be no worse than what now exists. This is not consistent with studies done by other agencies.

1.4.1.1.4 Flooding

The approximate crest elevations proposed by the DW Project would meet the height criteria for the reservoir side only. The crest elevation required to prevent the river/slough side embankments from being overtopped due to wave action on the design river flood elevation would not be met along the entire perimeter. The DW Project-proposed approximate crest elevations are therefore considered unacceptable.

1.4.1.1.5 Settlement

Large settlements of the embankments will occur during initial construction and continue throughout the life of the structure. The DW Project indicated that settlement occurring during construction would be compensated by placing additional fill. The DW Project does not indicate if and how the embankments will be raised in the future to maintain the current height, and the associated costs with this operation. Subsidence of the reservoir islands will reduce because of the change in use of the area. Severe cracking of the embankments will occur during construction and over the life of the project. The DW Project indicates crack repair and defensive design measures will need to be determined during final design.

1.4.1.2 Recommendations

- Factors of safety for sliding toward the river/slough do not meet the slope stability criteria. The DW Project recognizes that improvements on some areas of the river/slough side are needed based on the current modifications being considered for the existing levees. The specific areas requiring improvements and what type of improvement will be determined during final design.

- The crest elevation required to prevent overtopping from wave action on the design river flood elevation will not be met along the entire perimeter. To avoid overtopping because of river flooding, crest elevations of embankments for both reservoir islands would need to be raised.
- To minimize inundation of adjacent islands because of seepage and to avoid potentially critical seepage conditions resulting from exceptional events such as power failures and seismic events, it is recommended that alternative means of seepage control be investigated.
- Attempting to reduce the probability of embankment failure during a seismic event will be both difficult and expensive and minor geometric modifications will not reduce the probability of embankment failure. Improved emergency response plans and measures including stockpiling critical materials and equipment should be developed.
- Embankment failure may lead to water quality degradation, property damage and other adverse impacts. The probability of failure can be reduced with appropriate design changes such as flatter slopes, wider crests, and higher embankments. These and other solutions leading to overall system improvement are feasible and should be part of subsequent work.

1.4.2 Risk Analysis

1.4.2.1 Findings and Conclusions

The probability of failure of the DW Project embankments is as summarized below:

- The operational risk of embankment failure is small compared to seismic and flood risks. The highest potential risk is expected because of overtopping during a flood.
- The probability of embankment failure during construction with release of water from an adjacent slough into the reservoir area is significant (greater than 50 percent), if construction proceeds too rapidly or without staging.
- The slough-side embankments have the potential for failure under the long-term loading condition. The risk of these failures is considered to vary from marginal to unacceptable.
- The study estimates that there is about a 5.5 percent probability in 50 years (0.11 percent annual probability) that the Bacon Island embankments will fail during future earthquakes. The corresponding failure probability for the Webb Tract embankments is about 8.5 percent in 50 years (0.18 percent annual probability).
- For all sections that overtop, the probability of overtopping failure during the 100-year-flood events was estimated to be 39 percent for a selected project life of 50 years (0.01 annual probability).

The consequences of failure of the DW Project are as follows:

- Overall, the impact ratings to infrastructure, economy, land use, and health and safety resources are generally medium to low because the rural areas affected have lower asset values than urban areas.
- Failure of the Bacon Island embankment because of operations or earthquake could impact the Discovery Bay Housing development.
- Water quality and water supply consequences are high in the event of an inward embankment breach and upstream salt-water migration.
- Fisheries consequences are also high in the event of an inward embankment breach because fish may be entrained inside the reservoir once the higher slough water starts to recede. The magnitude of impact will vary depending on the fish species and life stage present at the time of the breach.

1.4.2.2 Recommendations

Solutions should be developed to enhance reliability of the DW Project and to meet design criteria. As part of these solutions, additional field investigation and laboratory tests should be carried out to address specific requirements of project reliability.

1.4.3 Fish Screens, Siphons and Pumping Stations

1.4.3.1 Findings and Conclusions

1.4.3.1.1 Design and Environmental

- Major problems with fish screens relate to cleaning requirements, screen mesh, perforated plate opening size, and screen area.
- The proposed DW Project will impinge and entrain fish, and cause localized predation losses.
- The project has long-term impacts related to maintenance and access improvements, including visual impairment, noise, recreation, and project lighting.

1.4.3.1.2 Risk of Potential Structural Failures

Embankment slumping, deformations, and lateral spreading may cause overstressing of the siphon and pump pipelines. The truss frame supporting the pumps on the reservoir side may experience strong ground shaking and deformation. In the case of such failure because of overtopping and if this takes place at the proposed location of pumps and siphons, these facilities could be severely damaged.

1.4.3.1.3 Operation and Maintenance

The proposed fish screens will be difficult and expensive to operate and maintain because of cleaning, access and structural problems.

1.4.3.2 Recommendations

The proposed DW Project designs for fish screens, siphons and pumping station structures appear to be deficient and risky. Facilities need to be consolidated for operation and maintenance reliability.

1.4.4 Design of Alternatives

1.4.4.1 Findings and Conclusions

1.4.4.1.1 Embankment Design

- The required crest elevation, based on a 100-year flood on the river, varies from 9.6 feet to 15.1 feet with an average of 10.2 feet on Bacon Island and 10.9 feet on Webb Tract. It is recommended that a variable crest elevation be used to minimize required new fill construction.
- A crest width of a minimum of 35 feet is recommended to accommodate traffic and future crest raises.
- Slope protection should be provided on both sides of the embankment to protect from erosion caused by the river and the reservoir.
- The existing river/slough side slopes have areas which do not met any design criteria for stability. Additional data gathering and analysis are needed to specifically identify areas that are unstable and require modifications. Additional analysis and design are required to determine what modifications could provide the needed stability and are environmentally acceptable. At this level

of study, to add the cost for this modification it was assumed the slopes would be cut back to a 4:1 slope above elevation 0 (MSL). As an alternative, the cost for 3:1 slope modification of the existing slopes was also included in this study.

- At this level of study, an island side slope of 3:1 down to elevation +4 and a slope of 10:1 below that elevation is recommended. Analysis that is more complete should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.
- To account for the cost of providing some means of protection from a piping failure that could occur from cracking because of settlement, an engineered filter zone was used for this study. Other alternatives for providing protection against such failure should be reviewed during the next phase of design.
- There is sufficient quantity of materials within the islands to meet the volume of either project alternative (the DW Project or the Re-engineered Alternative). The material in the borrow areas is expected to be saturated and will require drying prior to compaction.
- The design and cost should include provisions for maintaining an acceptable crest elevation resulting from settlement.
- Safety factors for the Clifton Court Forebay embankments will be higher than those computed for Bacon Island and Webb Tract because of different foundation conditions. Therefore, steeper slopes could be used for embankment modifications on Victoria Island.

1.4.4.1.2 Structures

Consolidated diversion facilities may offer a better solution. Future design should consider the level of operations and maintenance that will be required for the intake facility as opposed to only looking at initial capital costs. Consolidated facilities using flat plate screen technologies appear to show promise for this application.

1.4.4.2 Recommendations

- Analysis that is more complete should be done during final design.
- The need for, and type of dewatering system should be evaluated during final design.

1.4.5 Climate Change Impact

1.4.5.1 Findings and Conclusions

Global warming and rising sea levels may add additional constraints for the embankments as designed, by increasing the potential for failure because of overtopping. The embankment crests may need to be raised to meet water level changes caused by this potential change in climate. Associated project costs may be higher than estimated. Instead of a 100-year design flood, a higher magnitude flood, such as a 300-year event, may become the controlling design criteria for embankments and structures, and would require that embankment heights be increased by 0.5 to 1.0 foot.

1.4.5.2 Recommendations

Further work is recommended to assess climate change impacts on the project.

1.4.6 Cost Estimates

1.4.6.1 Findings and Conclusions

- The estimated total capital cost of development of the Re-engineered DW Project is $662 million. Depending on the extent of cost variations due to various factors such as changes in design, unit costs, site conditions and construction methods, this cost may increase to $1.1 billion.

- The Victoria Island Reservoir alternatives with connection to Clifton Court have higher capital cost of development compared to the Re-engineered DW Project because of the costs of raising Highway 4 and constructing a connection to Clifton Court.
- Annual operation and maintenance cost estimates for the Re-engineered DW Project and the Victoria Island alternatives are approximately $8.4 million.

1.4.6.2 Recommendations

The desired level of protection and physical design of the DW Project should be integrated through risk analysis to determine a realistic cost for developing it.

CHAPTER 2 – HYDROLOGY

2.1 Introduction

The Sacramento-San Joaquin Delta is a unique area situated at the confluence of the Sacramento and San Joaquin Rivers, which collectively drain about 43,000 square miles of watershed before discharging into the San Francisco Bay. The Delta occupies an area more than 1,100 square miles, including over 700 miles of waterways. The Pacific Ocean influences delta hydrology, and tidal river stages vary by more than 5 feet near Pittsburgh to less than a foot near Stockton. During tidal cycles, river flows also vary both in direction and quantity. Major factors influencing high water stages are high flows, high tides, westerly winds and low barometric pressure. The highest water stages occur in the winter, usually from December through February.

2.2 Recorded Flow And River Stage Data

Tidal cycles result in variations in river stages throughout the Delta. This tidal influence is even transferred in 1 to 2-feet stage variations in the Sacramento River near Hood. At the confluence of Sacramento-San Joaquin Rivers, the average tidal flow is approximately 170,000 cfs. In contrast, the average winter outflow at this location is 32,000 cfs and the average summer outflow is 6,000 cfs.

Recorded flow and river stage information used for diversions through structures was obtained from DWR California Data Exchange Center (CDEC) Database and Interagency Ecological Program (IEP) recording stations. The following stations were used in the hydraulic analysis for structures. The locations of these stations are shown on Figure 10.

- San Joaquin River at Andrus Island (Station ID. RSAN032)
- Middle River at Howard Road Bridge (Station ID. MHR)
- Old River at Bacon Island (Station ID. BAC)
- Old River at Byron (Station ID. ORB)
- Middle River at Tracy Boulevard (Station ID. MTB)

2.3 Storage Capacity

Reclamation/DWR's estimation of storage capacities of the reservoir islands was based on their respective areas at different contour intervals. Five-foot contour maps for Webb Tract, Bacon Island and Victoria Island were developed from DWR topographic surveys carried out in June 2001 and used to estimate storage capacities. Assuming storage to an elevation of +4 (MSL), the estimated storage capacity of Webb Tract was evaluated to be 102 thousand acre feet (TAF), while the storage capacities of Bacon Island and Victoria Island were evaluated to be 115 TAF and 108 TAF, respectively, as shown in Table 1, Appendix C.

2.4 River Flood Height

Two events of maximum river flood heights were considered for this study: river water heights resulting from a flood having an average recurrence interval of 100 years (100-year flood) and from a flood having an average recurrence interval of 300 years (300-year flood).

The 100-year flood level information was obtained from the report titled "*Sacramento-San Joaquin Delta Levee Rehabilitation Study, CALFED, September 1998*"[4], which was based on information published by the Federal Emergency Management Agency (FEMA). Projected flood elevations at various stations along the islands are shown on Table 2, Appendix C. The CALFED report states that in the event such a flood occurs, river water level elevations in the vicinity of Webb Tract could range from 6.6 to 7.1 feet above MSL, while for Bacon and Victoria Islands, these elevations could range from 7.1 to 7.4 and from 7.3 to 7.7 feet above MSL, respectively. These river water-level elevations, in conjunction with an appropriate freeboard and wave runup and setup, were used to evaluate the design elevations of the embankments along the perimeter of the reservoir islands.

The 300-year flood level information was obtained from data published by USACE. This information was utilized to evaluate the height and construction costs of embankments that would be required to minimize the potential for overtopping due to water level elevations resulting from climatic changes. The 300-year flood elevations are about 0.5 feet above the 100-year flood elevations.

2.5 River Wave Height

Riverside slopes of the island embankments are subject to the action of wind-generated waves and boat wakes. Wave heights recorded at various survey stations along the boundaries of Webb Tract and Bacon Island were obtained from the previously mentioned CALFED (1998)[4] report. At Webb Tract, the recorded wave heights varied from a maximum of 8.2 feet to a minimum of zero, while at Bacon Island, the recorded wave heights ranged from a maximum of 2.8 feet to a minimum of zero. Wave heights are provided in Table 2 of Appendix C. Wave heights on Bacon Island do not account for the adjacent Mildred Island being flooded, and should therefore be recalculated.

2.6 Reservoir Losses

The water stored in the planned reservoir islands would be subject to losses by evaporation, seepage through soil, leakage through gates, and onsite water usage related to project operations. Losses because of evaporation, infiltration, and seepage are of particular significance for the planned project, while losses resulting from leakage through gates and onsite water usage are expected to be relatively minor.

Evaporation: Evaporation losses were estimated for the Delta reservoir surfaces in operation modeling studies and were accounted for in operational runs. SWRCB permit allows topping off the reservoirs with additional diversion during summer. (See draft report on operations studies, February 2002.)

Seepage through Soil: Information on seepage losses is presented in Section 4.4 under Seepage Analysis. The DW Project proposes an interceptor well system to return seepage water going under the embankment onto the reservoir islands. Information on risk of failure because of seepage and loss of water to adjacent islands through seepage is presented in a separate report on Risk Analysis (URS Corp., 2001).

Leakage through Structures: Siphon stations and pump stations proposed by the DW Project are laid over the embankments and, therefore, no leakage losses are expected. However, leakage losses would occur through the gates of the Integrated Facilities in the re-engineered alternative. Leakage of water through a reservoir inlet/outlet gate is a function of the perimeter of the gate, the type of seal, and the differential head at the gate. Loss of water from the reservoir occurs when the water level in the reservoir is at a higher elevation than that within the inlet/outlet channels. This type of water loss cannot be controlled but is expected to be a relatively small.

Onsite Water Usage Related to Project Operations: This comprises onsite water use for purposes related to project operations such as sanitary and drinking purposes, cooling water for generators or other machines if applicable. This water use is not considered significant.

2.7 Reservoir Filling And Emptying

Reservoir filling and emptying operations play a critical role in influencing the integrity of the embankments.

Reservoir Filling - The initial filling constitutes the first test of a reservoir to perform the function for which it was designed. Keeping this in mind, the performance of the proposed reservoirs should be monitored during initial filling. To facilitate this, the rate of filling should be controlled to the extent feasible in order to allow as much time as needed for a predetermined monitoring program to take place. In addition to observing the conditions within the reservoir islands, the monitoring should ensure that filling these islands do not result in the development of adverse conditions at neighboring islands. The rate of filling should be such that the potential for erosion damage within the reservoir islands is minimal, i.e.; the velocity of water entering the reservoir should be controlled at the inlet gate. An optimal rate of filling should be established during final design in accordance with the above-mentioned criteria.

Reservoir Emptying - The rate at which a reservoir is emptied significantly influences embankment stability. Emptying a reservoir too fast can cause erosion, as well as trigger instability of the embankment slopes due to the development of a ‘rapid drawdown’ loading condition. This loading condition develops when the reservoir water level is lowered faster than the embankment ability to drain, resulting in a rapid decrease in the resisting force due to removal of the buttressing effect of the reservoir water. A suitable discharge rate for reservoir emptying operations should therefore be chosen to minimize erosion and to ensure that the stability of the embankment slopes is not compromised.

CHAPTER 3 – PROJECT GEOLOGY

3.1 General Geology

The Sacramento-San Joaquin Delta is part of the Central Valley geomorphic area, a northwest-trending structural basin separating the primarily granitic rock of the Sierra Nevada mountains from the primarily Franciscan Formation rock of the California Coastal Ranges. The Delta lies in an area underlain by 3- to 6-mile-thick sedimentary deposits, the majority of which accumulated in a marine environment from about 175 million years to 25 million years ago.

Since the late Quaternary time, the Delta has undergone several cycles of deposition, non-deposition and erosion, resulting in the accumulation of a few hundred feet of poorly-consolidated to unconsolidated sediments. Peat and organic soils began to form about 11,000 years ago during a rise in sea levels. This rise created tule marshes that covered the majority of the Delta. Repeated burial of tule and other vegetation growing in the marshes led to the formation of peat.

During the cycles of erosion and deposition, a number of rivers were entering the Delta from the north, northeast and southeast. These included the Sacramento, the San Joaquin, and the Mokelumne Rivers. As the rivers merged, they formed a complex pattern of islands and interconnecting channels. The river and channels were repeatedly incised and backfilled with sediments with each major fluctuation. Concurrent subsidence and tectonic changes in the land surface complicated these processes.

Debris produced by hydraulic mining during the gold rush of the mid-1800s disrupted the natural depositional history of the Delta. Hundreds of thousands tons of silt, sand and gravel were washed from the Sierra Nevada mountains into the Delta, which filled the stream channels and triggered flooding.

3.2 Topography

Existing perimeter levee cross sections completed for CALFED under the CALFED Levee Rehabilitation Study (1998) were used in the embankment analyses and computation of embankment quantities. Aerial photography was conducted in June 2001 and maps were prepared for Webb Tract, Bacon Island, and Victoria Island. These maps show 5-foot contours and were based on California Coordinated Zone 3, NAD83 and vertical NAVD88 systems. Ground crews surveyed the elevations of existing levee crowns every 25 feet on Webb Tract and Bacon Island. These elevations were included in the maps prepared for the islands. The new topographic maps were used to determine the area capacity curves for the reservoir islands, the locations and layouts of proposed facilities, evaluation, design, and analysis of facility components.

3.3 Field Investigations

Numerous investigations, laboratory testing, and analyses have been performed over the years within the Delta by DWR, USACE, Reclamation, and several engineering firms. Prior to field investigation, readily available and published reports were reviewed (Appendix A). Specifically, the geotechnical investigations conducted by HLA (1989) and URS(2000) were relevant to this study. To supplement and update the available information, a subsurface exploration program was organized jointly by DWR and Reclamation during the summer 2001. The exploration comprised drilling nine borings with Standard Penetration Test (SPT) sampling and sixteen Cone Penetration Test (CPT) soundings within Webb Tract and Bacon Island. The new investigations were not directly used in the analyses used in this study.

The CPT soundings were done by Reclamation between May 23 and 31, 2001. Of the sixteen soundings, eight were on Webb Tract and the rest on Bacon Island. At Webb Tract, the maximum CPT depth was approximately 108 feet below existing ground surface, while at Bacon Island, the maximum

CPT depth was approximately 128 feet below existing ground surface. The process involved hydraulically pushing a truck-mounted CPT probe (having approximate diameter of 1.4 inches) vertically down into the soil. For each sounding, a subsurface soil profile was developed based on interpretation of the cone tip resistance and the friction developed along the sleeve of the cone through its continuous advancement through the soil. The soil profile and other relevant subsurface information obtained during each cone penetration test were recorded in the form of a computer-generated graphical log. Copies of the CPT logs are provided in the referenced Reclamation Draft Geologic Data Package (2001).

The borings were drilled by DWR, between September 10 and 21, 2001. Four of the borings were drilled on Webb Tract to a maximum depth of 61.5 feet below existing ground surface The other five borings were at Bacon Island, to a maximum depth of 76.5 feet below existing ground surface. The holes were drilled using a truck-mounted hollow-stem auger drill rig having an approximate auger diameter of 8 inches. Disturbed but representative soil samples were collected at various depths. The samples were obtained by using a Standard Penetration Test (SPT) sampler, which was driven into the soil using a 140-pound automatic hammer dropped through a 30-inch free fall. The number of hammer blows required to advance the sampler through three 6-inch penetration intervals was noted. The samples were logged, geotechnically classified, and bagged for transportation to DWR's laboratory. For each boring, a detailed log containing information on the sampling interval, SPT blowcounts, geotechnical descriptions of the soil, and other relevant field data was prepared. Copies of the boring logs and other existing DWR geologic logs for the reservoir islands are presented in the referenced DWR Geologic Data Package (2001).

The results of Reclamation/DWR's field investigation and review of subsurface soil information presented in the referenced reports indicate that the site profile within the interior of the two islands (Webb Tract and Bacon Island) is reasonably consistent. Soft, organic clays and peats at the surface are underlain by fine-grained silty sand, which in turn is underlain by stiff clay and silt, and at greater depths, by medium to coarse-grained sand. The combined thickness of organic clays and peat generally ranges from a minimum of approximately 10 feet to a maximum of about 30 feet. Along the levees, an approximately 10-foot thick layer of fill consisting primarily of sand and clay soils with some organic content overlies the peat soils.

3.4 Seismicity

The Delta is located in a region of significant seismic activity. Numerous moderate to large earthquakes, some accompanied by surface rupture, have occurred in this region. A number of active fault sources have been identified within 62 miles (100 km) of the project islands. The major strike-slip faults such as San Andreas and Hayward are over 35 miles away. Small local faults and blind thrust faults are located in the immediate Delta area. The lack of reported severe earthquake-induced levee damage in the area indicates no significant earthquake motion has occurred in this immediate area since the construction of the levee system approximately a century ago.

A study of the seismic vulnerability of the Sacramento/San Joaquin Delta Levees[2] was conducted by the Seismic Vulnerability Sub-team of CALFED's Levees and Channels Technical Team in 2000. Peak horizontal acceleration versus return period curves developed during the Probabilistic Seismic Hazard Analysis (PSHA) carried out as part of this study are presented in Figure 8, Appendix C. A study of PSHA for the Tracy Fish Test Facility was done by Reclamation in 1999. This study provided preliminary earthquake hazard information for use in the feasibility-level studies for the Tracy Fish Test Facility, which is less than 10 miles south of Bacon Island. The peak horizontal accelerations for 10 to 10,000-year return periods based on the PSHA are also shown on Figure 8, Appendix C. Peak horizontal ground acceleration for a 500-year return period from this study is 0.31g, slightly higher than the CALFED study.

Dam embankments have been historically designed to withstand the Maximum Credible Earthquake. Current design practice is tending towards a probabilistic approach where the structure is designed based on an acceptable risk, where risk is defined as the probability of the load times-given by the probability of failure (given the load) times the consequences. The cost to design the proposed structures to withstand the maximum credible earthquake could be very high for the relatively low consequences.

Reclamation/DWR recommend that at a minimum, the project be designed to not fail under the loading for the 475-year return period (1997 Uniform Building Code for normal non-critical structures). This is also the earthquake loading used for the DW Project. During final design, a study should be done to determine, based on risk and economic considerations, to what additional earthquake loading the proposed structures should be designed.

3.5 Borrow Material

3.5.1 DW Project

The DW Project assumed that materials for modifying the embankments would come primarily from sand deposits within the islands. It was assumed that these sandy materials, with minor amounts of peat mixed in, would be used in the modification. The HLA explorations indicate that in general, the sand deposits exist beneath a layer of peat and organic soil approximately 10 to 15 feet deep. More recent field investigation by Reclamation/DWR revealed that the approximate thickness of this overlying layer of peat and organic clay varied, and was as deep as 30 feet in certain areas. To access the sandy materials, this overlying layer must first be excavated and removed from the borrow areas. The 2001 EIR/EIS provided the borrow requirements (see Table 3, Appendix C) and stated that borrow pits would need to be at least 800 feet away from the toe of the embankments to minimize the potential for seepage problems.

The DW Project proposed to place saturated material directly on the embankments without first drying the borrow pits The material would be dried in place and then compacted (letter to DWR from DW Project dated January 22, 2002). This could result in construction equipment getting stuck, lower density and strength of embankment materials, increased construction pore pressure problems, and increased potential for piping paths.

The DW Project assumed rock for riprap would come from the Dutra-McNeer Quarry or the Basalt quarry of Syar Industries and would be barged to the construction site.

3.5.2 Reclamation/DWR Re-engineered Project

Reclamation and DWR also assumed most of the material for constructing the embankments would come from the sand layers within the islands . The expected volume of earthwork is discussed in Chapter 7. Based on our review of available geotechnical information, it appears that there is a sufficient quantity of materials within the islands to meet the earthwork volume required in either project alternative, that is, the DW Project and the Re-engineered Alternative.

To obtain the required borrow material; the overlying peat layer must be excavated. It is assumed that the excavation would be carried out in strips or zones and the excavated peat would be used to backfill the exhausted borrow pits as construction progresses. Backfilling the borrow pits with the peat will ensure that the underlying sand layer is not in direct contact with the reservoir, thereby reducing the potential for seepage through this sand layer.

The interior of the islands is at or below mean sea level and the ground water table. The material in the borrow areas is therefore expected to be saturated and a dewatering system will probably be necessary to facilitate excavation. The need and type of dewatering system should be evaluated during final design. The excavated material will require drying before being used in final construction. There is sufficient area on the islands to spread the soil material to allow for natural drying. The requirements for excavating, dewatering, and drying the borrow materials should be determined before final design. However, this operation will add to the overall construction time and will also increase the project cost, because of the extra handling of the materials.

It is also recommend that at the time of initial construction, material be excavated and stockpiled for future use such as backfilling to maintain crest elevation (see Section 4.5, Settlement). The stockpile could also be used for emergency repairs, sliding, cracking, or slumping due to an earthquake.

Reclamation and DWR assumed riprap, bedding for riprap, sand filter, and gravel road surfacing would be acquired from commercial sources.

CHAPTER 4 – EMBANKMENT DESIGN

4.1 Review of DW Proposed Design

4.1.1 Crest Elevation

The DW Project proposed a constant crest elevation of 9 feet above mean sea level (MSL). The URS review for the 2000 EIR estimated wave runup and setup in the reservoir (Table 4, Appendix C) using the method in USACE 1984 Shore Protection Manual. The project assumed 60-mph winds and an effective fetch of 3.15 miles for Bacon Island and 2.83 miles for Webb Tract. This design did not account for increased reservoir elevation from the design storm event occurring directly over the project area. The method used was an industry standard and results were in the same range as the DWR estimates discussed below.

Freeboard was calculated by DWR in Bulletin 192-82. Wave height calculations varied with wind speed and open water dimensions and ranged from zero to 8.2 feet. Most wave heights ranged from 1.7 to four feet in the channels and seven to eight feet in more open areas like Franks Tract. Based on a 50-mph wind speed, a 4-foot wave height was calculated for open water ranging from 4,000 to 6,000 feet on the riverside. For river-side open water ranging from 2,000 to 4,000 feet, wave heights were 1 to 2 feet, respectively. There was no indication of slope or slope protection used in the DWR Bulletin analysis. The 100-year flood or the design flood event elevations and wave heights from the *"CALFED Long-term Levee Protection Pla*n" (1998) were summarized by embankment station (Table 2, Appendix C). Flood elevations varied from 6.6 to 7.4 feet above MSL. Crest elevations required to prevent overtopping varied from 9.6 to 15.1 feet with an average of 10.2 feet on Bacon Island and an average of 10.9 feet on Webb Tract (Table 2, Appendix C).

Reclamation and DWR recommended that the height of the new structures should be the larger of the two following criteria:

1. The normal +4 feet MSL reservoir water storage elevation plus wave runup and setup on the reservoir. If wave runup is less than 3 feet, then a freeboard of 3 feet should instead be added to the water storage elevation.

2. Water surface elevation of the design flood event on the riverside plus the wave runup and setup. If wave runup is less than 3 feet, then a freeboard of 3 feet should instead be added to the water surface elevation of the design flood event.

The DW Project-proposed crest elevations (approximately 9 feet MSL) that would meet the height criteria for the reservoir side only. The crest elevation required to prevent wave action resulting from the design flood event from overtopping the entire embankment perimeter on the river/slough side would not be met. For Webb Tract, 46,000 linear feet or 68 percentof the embankment would be overtopped and 16,000 linear feet or 26 percent would come close to being overtopped during the 100-year flood. For Bacon Island, 26,800 linear feet or 36 percent of the embankment would be overtopped and 32,400 linear feet or 43 percent would come close to being overtopped during the 100-year flood. A variable crest elevation of 9.7 to 15.1 feet was recommended to prevent this overtopping (Table 2, Appendix C). A variable crest elevation used instead of the maximum for the entire length of the embankments would reduce required fill volume. A final design re-evaluation of wave runup and setup was recommended especially for Bacon Island to account for the recently flooded and adjacent Mildred Island. Where the required crest elevation is greater than the average, consideration of parapet walls similar to the jersey barriers in Franks Tract was recommended. The higher crest elevation would result in increased quantities over what was estimated in the DW Project. In addition, overbuilding because of settlement of underlying soils would require additional fill. It was also recommended to include quantity and cost estimates for adding materials to the embankment to maintain the design embankment height over the life of the structure.

4.1.2 Crest Width

The URS analyses used a 35-foot crest width for the proposed embankments. The 2001 Final EIS indicates that the embankments would be initially constructed with a 30-foot wide crest and have a final crest width of 22 feet after raising because of settlement. In a telephone conversation with Reclamation and DWR on March 14, 2002, DW stated that their design intent was to have a 35-foot total crest width at the end of initial construction. The width of embankment would be potentially reduced to 22 feet in later years depending on the amount of long-term settlement that occurs and the resulting needed crest raises. The crest width needs to provide for two-way traffic for construction, maintenance, and to facilitate future fill placement to maintain the crest elevation. Reclamation/DWR recommends that a final 35-foot width be used to facilitate long-term maintenance and repairs.

4.1.3 Embankment Slopes

The new embankments proposed in the DW Project would not alter the existing river/slough-side slopes. Two configurations for the reservoir/island side slopes are proposed: a constant 5:1 slope, and a dual slope of 3:1 or steeper from the crest down to elevation +4, underlain by a slope descending at 10 to 1. Figure 13 in Appendix B illustrates these two proposed embankment configurations. The 2001 EIR/EIS indicates that more detailed studies would be done during final design to refine what slopes are needed in specific areas. These river/slough slopes are technically not acceptable as discussed in the static stability analysis sections.

4.1.4 Zoning

4.1.4.1 Slope Protection

The DW Project proposal indicates slope protection would be placed on the island side embankment slopes to control erosion due to winds and wave action. The 2001 EIR indicates conventional rock revetment, similar to existing exterior slopes or other conventional systems such as soil cement or a high-density polyethylene liner would be used. The July 2000 EIR indicates detailed analysis would be done during final design to size the slope protection for each section of the embankment. The DW Project proposal indicates geotextile would be used beneath riprap. As an alternative a typical sand, gravel and cobble bedding layer can be used.

Reclamation/DWR recommends providing slope protection for both sides of the embankment to protect from erosion caused by the river and the reservoir. On the reservoir side, the riprap should extend from the crest down to the island floor for the 5:1 slopes and to the slope break for the dual slope. Below the slope break the slope is flat enough so that slope protection is not necessary. On the river/slough side, the riprap should extend from the crest to the low-water mark (El. -2). For this study, 2.5 feet of riprap on 1 foot of bedding was considered. During final design, the size of the riprap should be re-evaluated to assure embankment stability.

4.1.4.2 Internal Zones

The DW Project proposal indicates that modification will be done with one type of material placed directly on the existing structure. The material would come from natural sand deposits on the islands, and from dredge-spoil sites. As discussed in the settlement section, the embankments are subject to cracking from differential settlement and there is a potential for piping. To minimize the potential for a piping failure, the DW Project proposes to place sand against the inside of the existing levees. No specific information regarding the type of material, thickness, or basis for where it would be placed are provided.

Reclamation/DWR recommends an engineered filter/crack stopper zone in the embankment to prevent piping failure because of differential settlement and cracking over the life of the project. Alternative methods should also be considered. For cost estimation purposes only, this study assumes that after the structure has been built to full height, a 3-foot wide trench would be excavated through the center of the embankment and backfilled with a sand and gravel filter.

4.1.5 Summary

Reclamation/DWR have the following conclusions about the general embankment design:

1. The DW Project proposed crest elevation (approximately +9 MSL) is unacceptable because a significant portion of the new embankments would be overtopped by wave action during the 100-year flood.

2. Reclamation/DWR recommends that a variable crest elevation, 9.7 to over 15, be used that would prevent overtopping from the 100-year flood. A variable elevation based on location in the Delta is recommended to reduce fill volumes. During final design, consideration should be given to using a parapet wall to provide the added height in areas where the crest elevation is greater than the average.

3. It is recommended that the originally proposed crest width of 35-feet be maintained at the end of original construction instead of reducing it to 22 feet to facilitate future crest raising because of settlement.

4. Discussion and recommendations on exterior slopes are included in Section 4.2.

5. The proposed slope protection on the exterior slopes is acceptable except that the geotextile beneath the riprap would not be recommended because of the potential for damage to it as a result of long-term settlement of the embankments. A standard sand and gravel bedding should be used.

6. Additional information and a detailed cost estimate are needed for the DW Project proposed methods to address the potential piping problem because of cracking and differential settlement.

4.2 Static Slope Stability Analysis

4.2.1 Method of Analysis

The DW Project evaluated embankment stability using the limit equilibrium method based on Spencer's method. This is the same method used by Reclamation/DWR. Other methods for evaluating static stability of embankments, such as finite element and three-dimensional analyses, are available, but the site geometry, loading, and other conditions do not warrant the use of these methods at this time. The limit equilibrium method is the industry standard for evaluating slope stability.

4.2.2 Design Criteria

4.2.2.1 Classification of Structures

The purpose of classifying the earthfill structures is to define the necessary design criteria. USACE, the state and Reclamation criteria and definitions listed below were reviewed. It was determined that the embankments should be classified as small dams and not levees.

- **USACE:** The USACE definition of a levee is "an embankment whose primary purpose is to provide flood protection and is subject to water loading for periods of only a few days or weeks a year." Since the purpose of this project is for water storage with storage of water for several months, the structures do not fall under the levee definition and these structures should be designed using earthen dam criteria.

- **State of California**
  - Per Water Code Sections 6002-6008 - Structures that are 25 feet or higher or have an impounding capacity of 50 acre feet or more fall under jurisdiction except structures not in excess of 6 feet high regardless of storage capacity or structures with a storage capacity less that 15 acre feet (Required to meet State of California dam design criteria).

  - Water Code Section 6004(c) – The levee of an island adjacent to tidal waters in the Sacramento-San Joaquin Delta, as defined in Section 12220, even when used to impound water, shall not be considered a dam and the impoundment shall not be considered a reservoir if the maximum possible water storage elevation of the impounded water does not exceed 4 feet above mean sea level, as established by the United States Geological Survey 1929 datum. If the exclusion of section 6004(c) is met, the structures do not fall under DSOD jurisdiction; otherwise the structures are under its jurisdiction as they impound more than 50 acre-feet of water.

- **Reclamation:** Reclamation would consider these structures embankments similar to small dams and design them according to its guidelines and standards.

4.2.2.2 Design Criteria Used by DW

The stability analysis done by URS (2000) for the DW Project mentions design criteria from various sources but does not specifically identify what was used . URS' report states that criteria should be "based upon significance of the project; the consequences of failure (economic, environmental and other); the jurisdictional status of the reservoir under California DSOD; and possibly other factors." The 1995 Draft EIR/EIS states "Levee improvements would be designed to meet or exceed state-recommended criteria for levees outlined in DWR Bulletin 192-82." The DWR bulletin addresses levees only and not structures used for water storage. Also, the bulletin provides no design criteria but identifies a typical levee section and states that specific designs are required for each site.

4.2.2.3 Recommended Design Criteria

Selection of design criteria needs to consider loading conditions and consequences of failure, reliability of shear strength parameters and pore pressures assumed, level of field investigations, variability of existing conditions, and quality of construction control. The following are recommended factors of safety for end-of construction, steady-state seepage, and sudden drawdown loading conditions. These values should be re-evaluated and adjusted based on the consequences of failure. Suitable design criteria based on consequences should be considered during final design. Table 5, Appendix C, summarizes the design criteria for the various loading conditions, for the specified material property and phreatic surface, from the various organizations, and the recommended values to be used for this project.

- **End of construction** - The design criteria for USACE levee structure, DWR, and Reclamation dams all require a factor of safety of 1.3. This will be the requirement used for this study.
- **Steady-state seepage** - This is a critical design criteria since embankment failure would release water and could have large economic consequences and potential loss of life including damage to adjacent islands. There is seldom much warning with these types of failures. Steady-state seepage is a loading condition that seldom develops in levees because water does not remain against the levees for sufficient time. A factor of safety of 1.5, higher than what is typically required for levees, should be used.

- **Sudden drawdown** - A sudden or rapid drawdown is not expected to happen at these structures because releases are limited by the pumping plant rate of 1.5 feet/day. However, a drawdown analysis should be performed because this rate of reservoir drawdown is still fast enough to prevent pore pressures from dissipating completely. Additionally, a sudden drawdown could occur if a breach is artificially initiated at a fixed location to minimize the damages of an impending failure. Required factors of safety vary from 1.0 for levees to 1.3 for dams. It is recommended that a factor of safety of 1.2 be used .

4.2.3 Geometry

4.2.3.1 Existing Configurations

Each of the islands has over 12 miles of existing levees that will be modified. Cross section data from the CALFED Levee Rehabilitation Study (1998)[4] were used to determine existing levee configurations. That study had 148 cross sections for Bacon Island and 69 cross sections for Webb Tract. Tables 6 and 7 (Appendix C) show average, maximum, minimum, median, and standard deviation values for some of the critical geometry for each of the islands.

4.2.3.2 Configurations of Existing Conditions Used in DW Analyses

The criteria used by the DW Project in selecting sections for analysis were existing levee height and soil conditions affecting stability. Two sections for each island were chosen, Stations 160+00 and 630+00 on Webb Tract and Stations 25+00 and 265+00 on Bacon Island. DW states that the sections are representative of the thickness of the levee fill from 6 to 10 feet, the thickness of the peat from 15 to 35 feet, 20-foot wide crest at elevation 8.5 feet, 2:1 slough side slope, and 4:1 island side slope. It was noted in the URS report (2000) that the most severe conditions that may be encountered might not have been analyzed and further data acquisition, additional analyses, and additional design configurations may be required during final design. As shown on the existing configuration (Tables 6 and 7 in Appendix C), the sections analyzed are representative of the average conditions and do not represent either the worst or the best condition.

Materials within the levees were modeled based upon explorations. URS modeled the existing levee fill as two layers. The upper layer is either a sand or silty clay with sand and is probably representative of materials used in more recent levee rehabilitation. Underlying that layer is a peat layer, which is probably a combination of native peat, and peat and peat-like material used to construct the original levees. The levee is underlain by a peat layer, a sand layer, and then a fat clay layer. URS assumed that the peat, which had fill placed on it, (topography showing material above elevation 0) had consolidated and was of a slightly higher strength. No parametric study is presented that would show the sensitivity of material properties or geometry.

4.2.3.3 Configurations of Existing Conditions Used in Reclamation/DWR Analyses

The analysis by Reclamation/DWR took a more generic approach to determine the sensitivity of the factor of safety to geometry. This will also help identify critical items where more data should be obtained. Variations in existing embankment height, slopes, and thickness of peat were used in this analysis. Based on the existing configuration data, the configurations shown in Table 8, Appendix C, were selected as typical sections for analysis.

4.2.4 Static Material Properties

As mentioned in Chapter 3, numerous investigations, laboratory testing, and other analyses of Delta levees have been performed. Based on a review of available reports listed in Appendix A, the material properties shown in Table 9, Appendix C, were used in these analyses. Also shown in the table are the material properties used in the HLA, URS and DWR analyses (1989-1990). The material properties used for the various analyses are not exactly the same and the variation could result in differences in computed factors of safety. Additional sampling and testing of the embankment and foundation materials should be performed during final design to refine knowledge of the material strengths.

4.2.5 Phreatic Surface

Water level fluctuations around the islands are affected by river flows and ocean tides. Tidal variations adjacent to the project islands are less than 6 feet. In the summer, the elevation range is typically from -1 to +3 MSL. Projected flood elevations and wave heights from the draft 1998 CALFED Bay-Delta Program at various stations along the islands are shown in Tables 2 in Appendix C. All analyses assumed an average waterway elevation of zero. An elevation of +6, slightly lower than the 100-year-flood level, was assumed in the DW Project design for the high-water level on the waterway side.

The groundwater under the islands fluctuates with waterway water levels, precipitation, and farming operations. Piezometers have been installed on the islands but have had limited monitoring. All studies assumed that groundwater was just below the existing ground surface when there is no reservoir.

The DW Project originally proposed to store water up to an elevation of +6 and this elevation was used in its supporting analyses. This elevation for storing water has since been reduced to +4. This 2-foot change in reservoir storage elevation would have minor effects on the phreatic surfaces assumed in DW's analyses and resulting factors of safety. Higher water level would tend to cause a higher factor of safety for the steady-state and rapid drawdown condition for sliding towards the island but lower factors of safety for sliding towards the slough.

The various water levels for the different conditions analyzed are shown on Table 10, Appendix C. All analyses assumed the phreatic surface was a straight line between the assumed water surface on either side of the embankment.

4.2.6 Results of Analyses on DW Proposed Embankments

4.2.6.1 Harding Lawson Associates Analysis (1989)

Preliminary geotechnical investigations and analyses were performed by Harding Lawson Associates (HLA) for the DW Project. The proposed plan at that time was to flood Bacon Island and Webb Tract to elevation +4. Eight cross sections, two for Webb Tract, two for Bacon Island, and four for other islands were analyzed for existing, end of construction, and long-term conditions. The sudden drawdown condition was not specifically analyzed, and it was assumed that the factors of safety were greater than the end of construction case because both conditions used undrained strength and there would be strength gain as the materials consolidated with time. Material properties used in the analysis are shown in Table 9, Appendix C, and were based on published data and laboratory tests of materials from the site. The proposed new embankment has a 10:1 slope on the island side from the crest. The cross sections analyzed did not include a widening of the crest, which was recommended. No changes to the slough/river side slopes were recommended. The factors of safety for the four cross sections on the islands of concern and the various loading conditions are shown in Table 11. All conditions except the long-term (steady-state) condition for sliding towards the slough meet the design criteria recommended in this report.

4.2.6.2 URS Greiner Woodward Clyde Analysis (2000)

At the request of Delta Wetland Properties, URS (2000) reviewed HLA's report to address concerns brought up by the State Water Resources Control Board (SWRCB). URS did additional static analyses and new dynamic analysis on the proposed new fills at Webb Track and Bacon Islands. The proposed new embankment configuration consisted of a crest elevation of +9 feet, a crest width of 35 feet, and a 5:1 slope on the island side. In addition, an island slope of 3:1 from the crest to elevation -3 feet and then a 10:1 slope was assumed for Sta. 630+00. It was assumed that water would be stored on the islands to elevation +6 feet, two feet higher than that used in the previous analysis. The stations used in the HLA analyses were used in this analysis. Existing, end of construction, long-term, and sudden drawdown static loading conditions were analyzed. Material properties used in the analysis are shown in Table 9, Appendix C, and were based on published data and laboratory tests of materials from the site.

The factors of safety for the four cross sections and the various loading conditions from the URS analysis are shown in Table 12, Appendix C. The factors of safety for surfaces sliding towards the islands side are lower than the previous analysis because of the new fill slope being a 5:1 instead of a 10:1. The factors of safety for sliding towards the slough/river side are also lower than the previous analysis and are primarily from modeling the river channel to be deeper, which reduces the resisting forces. The factors of safety for the end of construction condition indicate failure will occur if the new fill is placed all at once. It was recommended that staged construction be carried out to allow the peat to consolidate and gain strength before adding more fill. This is consistent with the construction practice for levees in this area. The long-term (steady-state) factors of safety meet or exceed all design criteria for sliding towards the island. Factors of safety for sliding toward the river/slough do not meet any of the design criteria. The study recognized these as marginal to unacceptable and recommended that the crest be wide enough so if sliding did occur there would still be enough width to prevent loss of reservoir until repairs could be made. The study also recognized that some sections had inadequate factors of safety for the sudden drawdown condition and revisions to the proposed configuration would be required in these areas.

4.2.6.3 Levee Rehabilitation Study (1990)

Delta levee overtopping and stability failures have occurred from the time the levees were first constructed. In 1989, DWR undertook a study to develop generic levee designs based on levee height, foundation materials, construction materials, and land use. The studies concentrated on island side geometry to meet long-term levee stability. Published laboratory data and back calculations of known failures were used to develop the material strengths shown in Table 9, Appendix C. The study developed a set of curves for island side slopes that would achieve a factor of safety of 1.3. In general, for low levee heights or small amounts of soft foundation soils, a 3:1 slope is sufficient. For large levee heights and thick layers of soft foundation materials, island side slopes of 7:1 or a berm with a slope of 13:1 may be required.

4.2.6.4 Reclamation/DWR Analysis

Reclamation/DWR's analyzed the stability of the perimeter embankment slopes using SLOPE/W, a computer program that uses limit equilibrium theory to compute the factor of safety of earth and rock slopes. Findings of the analysis are discussed below. Printouts of the computer analysis are included in Appendix E.

4.2.6.4.1 End-of-construction

Rapid placements of fill on peat material have historically failed because of the low strength of these materials and generation of pore pressures. The standard construction practice on soft foundations is staged construction, placing small amounts of fill at a time and waiting until the soil consolidates and gains strength after pore pressures dissipate. Analyses done by URS indicate factors of safety below 1 for the end-of-construction if the fill were to be placed all at once. As is proposed by the DW Project, Reclamation/DWR would recommend construction be done in stages over several years. The DW Project indicated it could take four to six years with three to four stages of construction to achieve the required consolidation and strength gain. For cost estimating, Reclamation/DWR assumed that fill would be placed over five years. As a part of final design, more in-depth stability analyses and laboratory testing would be needed to determine the actual fill placement rate. In addition, extensive monitoring during construction would be required and adjustment in volumes may be needed.

4.2.6.4.2 Steady-state Condition with Sliding towards River/Slough

The analyses done for the DW Project and confirmed in the Reclamation/DWR analysis (as shown in Table 13, Appendix C) indicate that the potential for sliding failure into the river/sloughs exists for the steady-state condition. The loading condition of water on islands has not been previously experienced by the existing levees, so no field data on their performance under this condition are available. The factors of safety of the existing levees are below the design criteria recommended in Section 4.2.2.3. This potential problem primarily exists where the channel is deep. The embankments with the existing slopes and a full reservoir have the potential to slide into the channels, which could cause unacceptable environmental damage, damage to floating structures, damage to adjacent levees, potential loss of life, and require expensive dredging to clean up. Loss of the reservoir may not occur because of the width of the embankments. The proposed modifications to the embankments by DW do not include modifying the river/slough side slopes. The costs for repairs and clean up of the slide mass should be accounted for in the overall cost of the proposed DW Project.

4.2.6.4.3. Steady-state Condition with Sliding towards the Island Side

The analyses performed for levees and for a storage reservoir indicate that slopes on the island side need to be 5:1 or flatter or be 3:1 with a buttressing berm. A continuous slope such as 5:1 generally requires a greater volume of material so a steeper slope with a buttressing berm is generally more economical. The slopes proposed by the DW Project fall within these ranges. Actual slopes should be based upon economics (quantity of fill required), staged construction requirements, and achieving a factor of safety of approximately 1.5. Reclamation/DWR concur with the DW Project that additional analysis should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.

4.2.6.4.4. Sudden Drawdown

The DW Project assumed that the factor of safety for sliding towards the river/slough would be the same as the steady-state condition. The failure surfaces do not intersect the reservoir and this is a reasonable assumption. Since this loading condition does not apply to levees, the design criteria recommended in Section 4.2.2.3 should be used. The reported factors of safety for sliding towards the river/slough, in some areas, are less than the design criteria, and the slopes should be modified as discussed under the steady-state loading condition.

4.2.7 Reclamation/DWR Re-engineered Design

4.2.7.1 Steady-state Condition with Sliding Towards River/Slough

Reclamation/DWR analyses indicate that the slopes of the embankments on the river/slough side could be flattened during initial construction to increase the factor of safety against a sliding failure. Careful construction control would be required to minimize environmental impacts. Flattening of the slope will require the centerline of the embankments to be shifted towards the island side and will increase fill quantities. However, excavated material can be used as new fill. The slopes needed to increase the factor of safety to the design criteria will vary depending upon depth of channel, thickness of peat, strength of peat, and height of embankments. Table 13, Appendix C, shows the Reclamation/DWR analysis for this loading condition with two variations of embankment height and peat thickness and varying peat strengths. Based on this analysis, it is recommended that a flatter slope, 4:1 above elevation 0 (MSL), be utilized on the river/slough side. During final design, analyses that are more specific should be done to determine actual slopes needed based on additional topographic data. Future designs should also consider alternative techniques that increase stability of river/slough slopes and have fewer environmental impacts.

4.2.7.2 Steady-state Condition with Sliding Towards the Island Side

Based on previous analyses and additional analyses, as shown in Table 14, Appendix C, Reclamation/DWR recommend a slope of 3:1 down to elevation +4 feet and a slope of 10:1 below +4 feet. Analysis that is more complete should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.

Figure 14 in Appendix B illustrates the perimeter embankment configuration proposed by Reclamation/DWR.

4.2.8 Summary

Reclamation/ DWR conclude that it is technically feasible to modify the existing levee structures and provide structures for a water retention facility. Based on the review of the DW Project analyses and conclusions and analyses done by others, Reclamation/DWR has made the following conclusions :

1. The method of analysis and assumptions used in the DW Project analyses are appropriate for this level of study.

2. The proposed embankments should not be considered levees and design criteria similar to small dams should be used instead of levee criteria. Water is stored against the proposed embankments for a longer time than existing levees. There are greater consequences of failure associated with proposed embankments than with levees now in the area.

3. Staged embankment construction over several years will be required. A five-year construction period is assumed, but further analysis will be required during final design to refine this estimate.

4. The existing river/slough side slopes in general do not meet the design criteria and in areas would be considered unacceptable. The DW Project did not recommend any modifications to these slopes and took the approach that a significantly wide enough crest would be used so that if sliding occurred the entire embankment would not fail and release reservoir water. This approach could cause environmental damage and safety issues, and result in costs that are not accounted for in the project.

5. For the re-engineered design, Reclamation/DWR assumed that the existing river/slough slopes would be to cut back to a 4:1 configuration above elevation 0 (MSL). This average configuration was used for computing quantities and cost at this level of design. As DFG has suggested modification of existing slough-side slopes to 3:1, cost of this modification was also estimated. More detailed studies should be done to determine specific areas of unstable slope and to determine what alternative designs could improve stability and reduce environmental impacts.

6. DW Project proposed slopes for the island side are acceptable for this level of study especially because it proposes to obtain more data and perform more detailed analysis to refine the slopes during final design.

7. A slightly different island side slope configuration was used for the Reclamation/DWR re-engineered design to obtain an average configuration for computing quantities at this level of study. Slopes of 3:1 down to elevation +4, followed by 10:1 below +4 were utilized. Analysis that is more complete should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.

8. All entities agree that additional data acquisition and analyses should be done during final design to refine required cross sections along differing reaches of the embankment.

4.3 Seismic Analysis

4.3.1 Design Criteria

4.3.1.1 Post Liquefaction Stability

Required factors of safety vary from 1.0 for levees to 1.3 for dams. A parametric analysis should be performed to determine the sensitivity of the factor of safety to assumed liquefied strength. The minimum acceptable factor of safety for a post-liquefaction stability should be 1.2 because there is variation in geometry and soil conditions at the site. The higher factor of safety is also recommended because of the increased consequences from the failure of the embankments relative to a levee.

4.3.1.2 Deformations

The general criteria recommended by Reclamation/DWR for evaluation of performance of low embankments are:

a. Deformation less than 6 inches - acceptable performance.
b. Deformation of 6 inches to 2 feet - marginal performance and extensive cracking could lead to a piping failure.
c. Deformation greater than 2 feet - there is a high potential for embankment failure due to piping or overtopping. Modifications in design are required.

4.3.2 Seismic Material Properties

The DW Project did not perform any post liquefaction analysis, so it did not make any assumptions on post-liquefaction soil strengths. Reclamation and DWR made the following assumptions in this analysis. Blow counts as low as 5 have been measured in the sands within the embankment and in the sand layer underlying the islands. Using Figure 9 (Appendix B), undrained residual shear strength of 100 to 400 psf should be assumed for the liquefiable materials in an analysis. If the peat is strained sufficiently to break the fiber bonds, then a residual strength should be assumed.

In URS dynamic analysis for the DW Project, the nonlinear dynamic behavior of the materials was modeled using an equivalent-linear method as proposed by Seed and Idriss (1970).

Triaxial shear tests on peat indicate that the peak strength is generally not mobilized in the soft material until the axial strain reaches 6 to 12 percent. This may cause the Newmark[15] deformation analysis (and the Makdisi-Seed analysis derived from it) to underestimate the deformation. Also, the results of vane-shear tests in peat indicate that the shear strength decreases dramatically once the fibers have been ruptured by rotating the vane less than one-quarter turn. If it is deemed that the shear strains caused by earthquake loading could be larger than those in the triaxial tests it will be appropriate to use vane-shear tests to determine whether that occurs in this peat.

4.3.3 Design Earthquake Motion and Response Analysis

The analysis performed by URS used two horizontal earthquake accelerations time histories, the 1992 Landers and the 1987 Whittier Narrows earthquakes. These time histories were modified to match the design earthquake response spectrum as proposed by the CALFED 1999 study. Two design earthquakes were used: magnitude (M_w) 6 and peak ground acceleration of 0.25g, and a magnitude (M_w) 7.7 and peak ground acceleration of 0.13g. These represent the local and distant controlling seismic events, respectively.

QUAD4M, a two-dimensional plane-strain finite element code, was used to model the embankment response under the earthquake load. Earthquake acceleration time histories were input at the base of the sand layer. It was assumed that liquefaction was not widespread and no significant changes in dynamic soil properties would occur. Average horizontal acceleration time histories of potential slide masses were determined. The critical slide masses were those identified for the static stability analyses. The average horizontal accelerations were then used in estimating deformations as discussed later.

The methods and assumptions used to determine time histories for use in the deformation analysis are considered appropriate.

4.3.4 Liquefaction Potential

The liquefaction evaluation presented in the Revised July 2000 EIR states that “a few pockets of potentially liquefiable soil deposits may exist in the levees and foundation soils. We believe however, that these liquefiable soil pockets are confined in limited areas and therefore are expected to have negligible adverse effects on the stability of the levees.”

The Final EIS (2001) goes on to state “soil borings indicate that some of the sand layers beneath the peat on the DW project islands have a potential for liquefaction, but levee reconstruction and island flooding would probably not increase nor decrease the potential for liquefaction and levee failure. Because the proposed levees are broader than the existing levees and broader levees distribute seismic effects over a larger area, total levee failure caused by substrate liquefaction would be less likely with the proposed levees than with the existing levees.” It should be noted however, that the actual DW Project-proposed crest width is not “larger” and this last statement is not necessarily true.

USACE's 1987 study “Sacramento-San Joaquin Delta Levees Liquefaction Potential” and the 2000 CALFED report identified Webb Tract as having high liquefaction potential (defined as 50 percent of the borings analyzed indicate liquefiable soils for a 5.5 magnitude earthquake and a ground motion of 0.1g). Bacon Island was identified as having moderate liquefaction potential (defined as 21 to 50 percent of the borings analyzed indicated liquefiable under the same loading). The USACE report also identified that levees on both Webb Tract and Bacon Islands had undergone prior earthquake damage. The 2000 CALFED report shows Webb Tract and Bacon Islands in a damage potential Zone II, medium to medium-high susceptibility, primarily where there are thick deposits of soft soils and liquefiable soils present.

Almost all explorations indicate that a thin layer of material at the top of the sand layer in the foundation is potentially liquefiable. This potentially liquefiable layer does exist under most of the length of the levees. Also, a portion of the levee fill is potentially liquefiable based on the material being loose, uncompacted sands particularly dredging spoils. Further explorations and testing are needed to define the exact limits of liquefiable materials.

4.3.5 Post Liquefaction Stability Analysis

A post liquefaction analysis was performed on the same sections used for the Reclamation/DWR steady-state analysis. Liquefied strengths of a small layer of material at the top of the sand layer were assumed to be 100, 200 and 400 psf. No other strength reductions were assumed. As shown in Table 15 of Appendix C for sliding towards the river/slough, if the liquefied strength is at least 200 psf, a post-liquefaction sliding failure is not expected to occur. As shown in Table 16 (Appendix C), for sliding towards the island, if the liquefied strength is at least 200 psf, a post-liquefaction sliding failure will probably not occur. During final design an in-depth analysis of CPT and SPT data should be done to verify that the minimum liquefied strength is 200 psf. Future analyses should also evaluate the potential loss of strength in the peat material due to straining beyond the fiber bond strength.

4.3.6 Deformations

URS estimated seismic-induced permanent deformations of the DW Project embankments using the Newmark Double Integration Method (1965) and the Makdisi and Seed Simplified Procedure (1978). The displacements for the DW embankment configurations were predicted to be 1.5 to 3.5 feet for the island side slopes and 3 to 4 feet for the river/slough side slopes for the four sections analyzed. The sections analyzed were the same as for the static analysis and do not reflect the most critical sections. The analysis also did not account for the loss of strength in the peat material due to straining beyond the fiber bond strength. Consequently, larger displacements could occur. The method and results are appropriate, provided the peat material is not strained beyond its peak strength. The embankments are not designed with defensive design measures like crack stoppers. The DW Project made no design changes to accommodate the predicted earthquake deformations.

The displacements predicted by URS would result in severe cracking and possible failure from erosion through cracks or an overtopping failure because of slumping and loss of freeboard. The proposed design by Reclamation and DWR would be less likely to fail during an earthquake because of the following reasons:

- Flattening river/slough slopes to increase the static factor of safety from 1.1 to 1.5 is expected to reduce the potential dynamically induced deformations in that direction.

- Increase in crest elevation will reduce the potential for an overtopping failure.
- The larger toe buttress will reduce the amount of deformations.
- If an engineered filter is added, it may act as a secondary design measure to reduce the potential for a piping failure because of dynamically induced cracking.

Although the proposed design changes by Reclamation/DWR will reduce the deformations and potential for a failure during an earthquake, some failures may still occur.

4.3.7 Seismic-Induced Seiche

URS evaluated the potential for a seismically induced seiche for the reservoir islands using the United States Committee on Large Dams 1995 method. The predicted seismic-induced wave is less than 1 foot. With over 5 feet of freeboard it is considered unlikely that such a wave would overtop these embankments.

4.3.8 Summary

The analyses indicate deformations in the range of 1.5 to 4 feet could occur during an earthquake. These deformations could lead to cracking and a potential piping failure as well as loss of crest elevation and overtopping. The 2000 CALFED report concluded, "attempting to significantly reduce seismic levee fragility will be both difficult and expensive, and that simply making relatively minor geometric modifications will not significantly reduce seismic vulnerability. Developing improved emergency response plans and measures (including stockpiling critical materials and equipment) is thought to have considerable merit, especially in the short term."

4.4 Seepage Analysis

4.4.1 Selection of Sections

URS modeled four sections: Webb Tract Stations 260+00 and 630+00 and Bacon Island Stations 220+00 and 665+00. Webb Tract Station 630+00 and Bacon Island Station 220+00 were deemed the more critical sections where seepage was thought to be more of a problem because of the following reasons:

1) Narrow channel width to adjacent islands
2) Thicker high conductivity sand unit
3) Thinner low conductivity unit overlying the sand unit

Webb Tract Station 260+00 and Bacon Island Station 665+00 were considered by URS to be less critical, or more representative of seepage for the islands as a whole. The URS reports also points out that the least critical sections are adjacent to wide water bodies like the San Joaquin River where there is no island nearby.

4.4.2 Method of Analysis

Two groundwater-modeling efforts have been performed to quantify DW Project seepage. Harding Lawson Associates (1991) used the FLOWPATH finite difference code to construct an areal (plan view) two-dimensional model and URS Greiner Woodward Clyde (2000) used the SEEP/W finite element code to construct vertical slice two-dimensional models.

All two-dimensional models have some limitations when applied to real world systems and thus both of the previously mentioned approaches have certain drawbacks. Areal 2-D models cannot capture variability with depth and vertical slice models cannot truly simulate the radial flow converging to the capture wells. The vertical slice approach of URS is considered more appropriate for the project. As with all models, the URS model is very sensitive to the porous media properties of the silt lining the channel beds. URS considered this property in sensitivity analyses and obtained good calibration with reality.

Reclamation/DWR did not perform any analysis for this study. Previous studies were reviewed and the URS study closely parallels the modeling concepts Reclamation/DWR would apply. Reclamation/DWR concluded that the URS model is a creditable model for assessing seepage.

4.4.3 Modeling

URS modeled three scenarios:

1) Existing conditions
2) Project with full reservoir (elevation +6 feet)
3) Project with full reservoir and interceptor wells

URS employed four constant head boundary conditions:

1) Slough: elevation 1 foot (a constant to approximate daily tidal transients)
2) Full reservoir: elevation +6 feet
3) Far field: calibrated in the existing conditions scenario to match observed water levels beneath the floor of the islands and then fixed for subsequent scenarios
4) Wells: a constant head along a line representing the interceptor wells approximating an average drawdown of the well's cone of depression

It can be argued that a specified flux (Neumann type) boundary condition should have been used to simulate the wells and that a head-dependant flux (Cauchy type) boundary condition should have been used for the far field. The former would simplify exploring various pumping rates. The boundary conditions used are probably fine.

Sensitivity analysis (Webb Tract Station 630+00 only) consisted of:

1) Increasing channel silt hydraulic conductivity from $1x10^{-6}$ to $5x10^{-6}$ cm/s,
   Effect: Produced flooding on the island. Does not match observations.
   Head under the levee increased. Seepage from channel increased by a factor of 2.5.
   Change to interceptor well pumping rate was insignificant
2) Increasing sand unit hydraulic conductivity from $1x10^{-3}$ to $5x10^{-3}$ cm/s,
   Effect: Head under the levee decreased. Seepage from channel increased by a factor of 4.
   Interceptor well pumping rate increased by a factor of 3.
3) Decreasing peat thickness on the island floor from 6 to 3 feet.
   Effect: Produced flooding on the island. This, however, does not match observations.
   Head change was insignificant. Change to interceptor well pumping rate was insignificant

The project is formulated with a reservoir elevation of 4 feet; therefore, the amount of seepage that will need to be intercepted to prevent flooding of adjacent islands will be somewhat less than that predicted by URS.

4.4.4 Soil Parameters

Table 17, Appendix C, shows the hydraulic conductivity values used by URS to model the four sections. The actual materials and material thickness vary by section (URS, Table 2.3.1, pages 2-26,2-27). These values are reasonable for the task. The literature gives conductivities on the order of 10^{-7} for compressed peat, which will apply under the embankments; however, compressed peat of low hydraulic conductivity will have little influence on total seepage. The hydraulic conductivity of the sand was derived from a pump test on Holland Tract and extended to Webb Tract and Bacon Island by grain size-hydraulic conductivity relationships. It appears to be low, but the D_{10} particle diameter indicates appreciable (greater than 10 percent) fines (percent of soil passing through US No. 200 sieve) which is sufficient to reduce conductivity considerably. However, even by taking the percentage of fines into account, a value nearer to $5x10^{-3}$ cm/s, the high value explored in the sensitivity analysis, is expected. More field measurements of hydraulic conductivity values are needed before final design.

4.4.5 Recovery System

Both models indicate water levels under adjacent islands will rise because of the DW Project. The DW Project proposes to use interceptor wells to control water levels. URS gives a pumping rate of 10 to 12 gpm/ft per well for 160-foot well spacing with local variations. It is prudent to assume the sand unit hydraulic conductivity of $5x10^{-3}$ cm/s from the sensitivity analysis applies, and the pumping rate will be roughly 30 to 36 gpm/ft per well for 160-foot well spacing.

As an alternative to high-maintenance interceptor wells, Reclamation/DWR recommends that interceptor drains at the island-side levee toe of adjacent islands be considered. Since some of the reservoir seepage is lost to the channel while en route to the adjacent island, drains would need to handle a lesser flow rate than wells on the reservoir islands. These drains could be constructed much like Reclamation's agricultural drains with a low ground pressure, high-speed trencher. The trench could be excavated, tubing installed, and envelope placed in one pass.
Drain features would be as follows: 1) perforated, corrugated polyethylene drain pipe (6 to 10 inch diameter), 2) pipe slope of 0.001 foot/linear foot, 3) pipe invert depth of about 9 feet (below peat),
4) graded gravel envelope or graded gravel filter, 5) clean-out riser at end, 6) manholes every 500 feet, 7) pumped sump about every 1,000 feet.

Drains can be cleaned with a water jet when needed. At locations where ground water contacts an organic carbon source such as peat, bio-fouling can be a severe problem. Both drains and wells are susceptible to bio-fouling. Limiting the oxygen source can reduce fouling. It is typically easier to eliminate aeration in a drain than in a well. Depending on how the pumps are distributed in the well system — for example, one pump per well or many wells manifolded to one pump — the drain system could have fewer pumps to operate and maintain. Easements from landowners would be required. This alternative may not be feasible because of the inability to obtain easements on the adjacent islands.

4.4.6 Piping

No quantitative analyses have been performed at this stage to evaluate the potential for a piping failure. This should be done during final design. The embankments have a relatively large horizontal seepage path with a relatively low head. For these types of conditions, the chance of a "blow-out" failure is considered minimal. There exists, however the potential for piping failure because of seepage through the embankments and through cracks (see Subsection 4.5). The DW Project does not include any specific measures to reduce the potential for a piping failure. Different alternatives are available to address this issue. For this study, DWR/Reclamation considered an engineered filter to reduce the potential for a piping failure.

4.4.7 Summary

Reservoir seepage is expected to cause water levels to rise under adjacent islands. Measures to control this seepage are needed. Interceptor wells through the embankments around the reservoir or interceptor drains on adjacent islands can be used to control seepage. Site-specific field hydraulic conductivity tests are needed to refine seepage estimates. A model to address seepage to drains should be developed.

4.5 Settlement, Subsidence, And Cracking

4.5.1 Settlement

The Delta levees are built on, and of, extremely compressible peat and soft clay materials that have caused ongoing consolidation and levee settlement. The amount of settlement has varied depending upon the thickness of the peat in the foundation and the height of the fill.

Table 18, Appendix C, provides the settlement for different fill and peat thickness estimated by HLA. The estimates were based on analysis of typical levee sections of the project islands and studies done on Delta islands by others. HLA also predicted that half of the settlement due to of placement of additional fill would occur within 2 to 3 months, another one-quarter would occur within 3 years, and the remaining would occur at a decreasing rate over 30 to 50 years. HLA also estimated that the fill required to compensate for settlement in the first year or two would be on the order of one-third to one-half the initial section (measured from original grade) and could approach two-thirds to three quarters over the life of the project.

Another settlement analysis was performed by URS using the one-dimensional consolidation tests HLA performed. URS' analysis was based on a section at Sta. 630+00 on Webb Tract, which has a layer of peat about 20 feet thick at the centerline and less than 10 feet of fill on the downstream slope. The predicted total settlement was 9 feet, which is consistent with the HLA study.

The USACE Sacramento District, October 1982 report titled "Sacramento-San Joaquin Delta California Documentation Report", provides information on settlement of the Delta islands. The study predicted that the levees would reach total settlement in about 150 years. The levees will reach 50 percent of their total settlement in about 18 years, and 75 percent of their total settlement in 42 years. They also recommended that for general levee rehabilitation no more than 12 inches of material be placed per month and no more than 3 feet of material be placed on natural peat in any one construction season.

Reclamation/DWR also evaluated settlement based upon HLA testing. In addition, settlement studies done in the 1960s on Jersey and Empire Islands were reviewed. Based on this, it is recommended that an overall increment factor (that is, a multiplicand) of 2.25 be used to estimate required fill volumes. It is not clear what or if any such factor was considered in the DW Project.

4.5.2 Subsidence

The Delta islands are subsiding, which has required periodic raising of the existing levees to protect them from overtopping. The subsidence is caused by several complex and interrelated factors including oxidation of peat, wind erosion, tectonic movement, compaction, consolidation, burning, and to a lesser extent anaerobic decomposition of the peat. A study done by the USACE in 1980 predicted the rate of subsidence on these islands to be about 3 inches per year. The subsidence could be as much as 12 feet for a 50-year design life of the project. Converting these islands to water storage facilities and eliminating agriculture use is expected to reduce the subsidence significantly, but will probably not eliminate it. URS assumed that the subsidence of the islands would be about 1 foot. Reclamation/DWR concur that loss of embankment height because of subsidence over the life of the structures will probably be minimal.

4.5.3 Cracking

Cracking has historically been observed in the Delta levees, and has caused failures. Placement of fill over soft compressible peat results in significant ground settlement as the peat consolidates under the fill weight. Differential settlement occurs in the embankments because of varying heights, periodic placement of toe berms and fill on the levee slopes and crests, and from downslope creep. Settlement of new embankment is expected to be larger at the existing toe than the crest because the new fill near the existing toe will be thicker than at the crest. This differential settlement has a potential to create tension zones and cracks in the embankments. Differential settlement and cracking occur where the embankment crosses old clay or peat filled sloughs. Cracks in the embankments may also occur because of shrinkage during the extended dry months. The greatest immediate danger because of cracking is the possibility of water penetrating the cracking system to flow freely through it and eroding the fill, leading to a piping failure. Various methods to protect against piping failure exist, including the use of engineered filters, downstream drainage systems, jetting material into the cracks during construction, and others.

The 2001 EIS indicates that sand would be placed against the island side of the levees to reduce the potential for a piping failure because of cracking. In addition, DW Project plans long-term monitoring and areas of levees showing distress would require maintenance such as the placement of additional fill or implementation of other erosion control measures. No specific details of design or configuration of the sand layer were provided by DW.

Reclamation/DWR recommends that adequate measures be taken to minimize the potential for a piping failure because of settlement and cracking. Various options are available to address this type of failure. An engineered filter zone was considered in this study in order to account for the cost of providing protection against such failure. For this study, it was assumed that after full-height embankment construction a 3-foot-wide trench would be excavated through its center and backfilled with a sand and gravel filter. Other viable alternatives should be reviewed during the next phase of design.

4.5.4 Summary

Significant settlement is expected during construction and over the life of the embankments. The amount of settlement will vary depending upon peat thickness and fill height. Analysis done by Delta Wetlands Properties indicates that additional fill is required, but the specific assumption has not been provided. Based on the above analyses Reclamation/DWR made the following assumption for this study:

- Increase initial fill volume by a factor of 2.25
- Construction will take at least 5 years
- Place an additional 25% of the fill volume 15 years after construction

The actual amount of additional fill required to compensate for the settlement should be estimated during final design. In addition, consideration should be given to overbuilding the embankments during initial construction and subsequent modification to ensure adequate crest elevation is maintained at all times and to prevent overtopping during the design flood event.

The DW Project did not mention any specific method that would control piping and prevent piping failure. Reclamation/DWR assumed that a sand filter, of some type and location, would be required to prevent a piping failure. For cost estimating purposes only, a vertical filter through the middle of the embankments was assumed. Additional measures should be identified and evaluated during final design.

CHAPTER 5 – STRUCTURES

5.1 Scope

Delta Wetlands Properties proposed new siphons with fish screens for diversion and new discharge pumping units on the reservoir islands (Webb Tract and Bacon Island). Some of the old siphons will be removed and the others will be upgraded on the reservoir islands, as well as the habitat islands, (Holland Tract and Bouldin Island).

If state and federal agencies decide to acquire the DW Project, they would operate it under public ownership standards and the terms and conditions of a permit issued by the State Water Resources Control Board (SWRCB). This permit may be revised through a supplemental environmental impact report.

With state and federal ownership, the re-engineered project may have the same reservoir islands and habitat islands as the DW project, but structures and levee systems may change. Of the two other project alternatives, one considers storing water at Bacon and Victoria Islands (i.e., Victoria Island replaces Webb Tract), while the other considers storing water at Webb Tract and Victoria Island (i.e., Victoria Island replaces Bacon Island). Victoria Island as a reservoir island provides for direct diversion to Clifton Court but requires siphons, pumping and a conveyance channel to transfer water there, in addition to integrated diversion and release facilities.

This chapter presents technical information on engineering design of DW Project structures and re-engineering needed to integrate diversion and release operations.

5.2 Design Criteria and Assumptions

5.2.1 General

Design Flows for Structures: Sizes of structural components are based on the following maximum daily diversions and releases permitted from Webb Tract and Bacon Island as stated in the SWRCB Permit for DW:

	Total	Webb Tract	Bacon Island
Diversions			
All islands combined			
Total max day	**6,000 cfs***	**3,000 cfs**	**3,000 cfs**
		(1,500 cfs – San Joaquin)	(1,500 cfs – Middle River)
		(1,500 cfs – False River)	(1,500 cfs - Santa Fe Cut)
Total monthly average	**4,000 cfs***		
Discharges			
All islands combined			
Total max day	**6,000 cfs**	**3,000 cfs**	**3,000 cfs**
		(1,500 cfs – San Joaquin)	(1,500 cfs – Middle River)
		(1,500 cfs – False River)	(1,500 cfs - Santa Fe Cut)
Total monthly average	**4,000 cfs**		

* Habitat Island diversions included

5.2.2 Siphons and Pumping

- Siphon operations are assumed to be under gravity flow, gravity and booster pumps combination, and pumping only.
- All inlets for siphons or pump intakes are submerged and pumping occurs under full-flowing conditions for siphons or intake penstocks.
- The assumed maximum size of the pumping unit is 500 cfs.

5.2.3 Fish Screens

- An approach velocity of 0.2 feet per second is assumed based on the USFW Delta Smelt Biological Opinion. The same exit velocity will be maintained at the screens during releases from storage.
- For smaller structures, the DW Project proposes barrel type screens requiring manual cleaning. For larger structures, vertical flat-plate screens are assumed for this preliminary phase of design. Further evaluation of different types of screens is needed for the more detailed design. Fish salvage is not considered and thus no salvage structures are included in fish screen structures.
- Fish screens are assumed to be either self cleaning or cleaning provisions will be provided.

5.2.4 Inlet/Outlet Structures and Flow Channels

- In flow situations, where relatively high velocities cannot be avoided, flow energy is dissipated by expanded wing walls, submerged hydraulic jumps or energy dissipation devices such as stilling basins and baffles.
- Structural design is based on non-eroding flow velocity conditions. Riprap protection is assumed in all sloping and bed areas of flow channels.
- For embankments around structures, embankment sections and Minimum Factor-of-Safety Criteria for slope stability design are adopted from the design criteria for reservoir embankments as shown in Section 4.2.2.

5.3 DW Proposed Structures

5.3.1 Description of Components

For diversion of water into islands, the reservoir islands will be provided with 64 new siphons with pumping units and fish screens. Also, 35 existing siphons on the reservoir islands and 18 existing siphons on the habitat islands will be upgraded with new fish screens. In addition, 773 seepage interceptor wells are proposed, to reduce seepage to adjacent islands. For release or discharge of water, a discharge station is proposed at each island, with the station at Webb Tract having 32 pumps and the one at Bacon Island having 40 pumps. Locations of the newly proposed intakes and discharge stations and existing siphons are shown in Figures 2 and 3, Appendix B. Layout plans and detailed sections of these structures are shown in Figure 11 (page 1 of 5 through page 5 of 5), Appendix B. The structural components, as proposed in the DW Project, are described below.

5.3.1.1 Siphons

The proposed DW Project plans to add 32 new siphons to each of the reservoir islands (64 siphons each). In addition, 28 existing siphons at Bacon Island and 7 existing siphons at Webb Tract will be modified for diversion of water for immediate use or storage. Each reservoir island will have 2 new siphon stations; each station containing 16 siphons spaced 40 feet apart.

Each new siphon would consist of the following components:

- An inlet with a fish screen submerged in the channel.

- A 36-inch diameter rigid pipe constructed along the river/slough side slope to the top of the embankment and then cut through the top of the embankment to the interior side slope. The pipe would be supported by six pilings.
- A 36-inch diameter rigid pipe installed from the top of the embankment along the interior slope to the interior of the island. The pipes would be connected at the top of the embankment and would be supported by concrete tracks on the interior slope. An expansion chamber would be connected to the interior pipe. The expansion chamber would allow the siphon pipe to expand from 36-inch diameter to 36-inch by 120-inch rectangular opening to disperse high velocity flows and reduce erosion. Riprap on the island floor would prevent erosion around the discharge end.
- A control valve, a meter, and a booster pump.
- A hinged flap gate to prevent backflow.

Guard piles will be constructed in the channel beyond the inlets to protect the siphon units. A standpipe will be provided on the higher elevation of each siphon for attaching the vacuum pump to start the siphon.

Each siphon station will include: (1) a boat dock, supported on 16 pilings (10 berths max) for use by maintenance personnel, (2) a 50-feet by 100-feet maintenance facility, (3) parking area, (4) living quarters or office space constructed on pile foundation, and (5) an equipment access ramp near the maintenance facility for access from the levee road. The station will occupy about 900 feet of the embankment and would cover about 3.4 acres.

During operation-start-up and shutdown, siphon units will be started and stopped sequentially in each station to avoid creation of bore waves and surges in adjacent channels.

5.3.1.2 Discharge Pumping Stations

Water will be released from each reservoir island through discharge pump stations. There will be one discharge station per island, each consisting of the following:

- 32 pumps on Webb Tract and 40 pumps on Bacon Island.
- A 36-inch diameter rigid pipe installed through the top of the embankment and along the exterior slope of the embankment down into the channel. The pipe will be connected to an expansion chamber that will allow expansion from 36 inches to a 36-inch by 120-inch rectangular opening. Guard piles on the Delta channel would protect the expansion chambers, and riprap on the bottom of the channel would protect against erosion.

The pumps will most likely be electric. Portable pumps will be used as standbys. The portable pumps can also be used as supplemental units to meet varying discharge requirements. The total releases will not exceed the maximum specified discharge rate of 6,000 cfs for both islands.

Each pump station will have a boat dock, a maintenance facility, living/office quarters, parking lot, and an access ramp like the facilities described under siphons, above.

5.3.1.3 Fish Screens

Fish screens will be installed at all siphon intakes on both the reservoir islands and the habitat islands. There will be 99 fish screens on the reservoir islands and 22 on the habitat islands. The screens will be barrel-type with a hinged flange connection at the water surface to allow rotation of the screens out of the water for cleaning and repair. The barrel will be enclosed by 304 stainless steel woven wire mesh (7 openings per inch in screen of 0.035-inch-diameter wire) and will include panels for access and cleaning the interior of the barrel. The ends of the barrel will be covered with cones made out of 5/32 perforated 304 stainless steel metal.

For cleaning, the screen modules will be raised out of the water and cleaned by high-pressure water or steam. Cleaning practices will follow the agreements as set forth in the Biological Opinions (BO) and incidental take permit issued by the regulatory agencies. The approach velocity of water is restricted to 0.2 feet per second in the BO. It is unknown if the design of fish screens stated above maintains the required velocity.

5.3.1.4 Embankments in Structures Area

Embankments around structures are in accordance with Chapter 4 of this report.

5.3.2 Analysis of Delta Wetlands Proposed Project

URS and CH2M HILL consultants carried out the following two types of analyses for the structures proposed in the Delta Wetlands Project:

- Fish screens, siphons and pumping evaluations
- Overall risk of project failure

Separate reports on these studies are available (URS, 2001 and URS/CH2M HILL, 2001). Summary information on Siphons/Pumping and Fish Screen Evaluations is presented in the following sections. URS' approach on overall risk analysis and evaluation of the consequences of Delta Wetlands Project is presented in Chapter 6.

5.3.2.1 Fish Screens, Siphons and Pumping Evaluations by URS/CH2M HILL

Operation, maintenance, structural stability and costs of the proposed fish screens, siphons and pumping units were evaluated. A separate report on these evaluations, compiled jointly by URS and CH2M HILL, is available (URS/CH2M HILL, 2001).

5.3.2.1.1 Scope of Work

The scope of work consisted of the following tasks:

- List fish screen design criteria as used in the DW Project.
- Identify potential environmental impacts (i.e., aesthetics and others) of the DW Project for existing and new pumps/siphons/fish screens along the perimeter of Webb Tract and Bacon Island.
- Identify and discuss operation and maintenance issues associated with the proposed DW Project's pump/siphon/fish screens.
- Identify and evaluate structural issues associated with the design, construction, and operation of existing and new pumps/siphons/fish screens systems as proposed in the DW Project.
- Discuss the failure or damage potential of existing and new pump/siphon/fish screen systems as proposed by the DW Project for earthquake, settlement, and flood conditions.
- Estimate the total operation and maintenance costs for the DW Project including long range costs associated with this proposal in case of structural damage or failures. The costs will be based on historical costs for similar facilities and adjusted for this proposal.
- Discuss the need and benefits of consolidated pump/gravity integrated facilities.

5.3.2.1.2 Analysis and Results

The proposed DW Project intake system has 64 new intakes with pipelines, pumps, screens, and structures, as well as 57 retrofitted intakes with screens and structures. In addition, DW has proposed 72 siphons and pumps for release of water. The applicability of this concept is unproven at this scale and may necessitate smaller and more manageable facilities. However, structural and hydraulic considerations should not be the only things modified. Functional cylindrical intake screens should be designed based on lessons learned from other installations and failures. The lessons learned from the previous poor performance or failure could prevent costly mistakes and redesigned facilities on the DW Project.

The proposed DW Project screen design has very few of the functional features. Specifically, the proposed design will be very difficult and expensive to operate and maintain for the following reasons:

- No automatic screen cleaning system.
- Poorly retrievable system – even when raised, it will still be subject to corrosion, and poor access for inspection.
- No monitoring system.
- Poor access to the screen.
- Dissimilar metals on pipe and screen.
- Woven wire screen (stainless steel is good but not resistant to bio fouling).
- Structural inadequacy.

Downsizing each diversion (to 40 cfs, for example) may result in a more manageable screen unit as recommended by DFG Criteria for Fish Screen Design. Therefore, the number and complexity of the intake pipes and facilities will increase four fold, or to approximately 248 new diversion structures and pipelines. An additional 72 pumping and siphon units will be required for release of water. Within this concept, it is recommended that each of these screens be designed and equipped with automatic cleaning systems, retrievable screen systems, cathodic protection, wedge-wire screens and monitoring systems. In addition, each screen should be able to withstand higher structural loads and pressures. The resulting cylindrical screens may be manageable on an individual level, but unmanageable when considered as a whole.

Designing major fish screen intakes for the DW Project must include considerations related to fisheries protection structural, hydraulic, and geotechnical issues. Operations and maintenance must also be considered to ensure a successful fish screen project. The design proposed in the DW Project does not satisfy these objectives and is therefore deficient or a risky design. Future design should consider the level of operations and maintenance that will be required for the intake facility as opposed to only looking at initial capital costs.

5.4 Integrated Facilities

5.4.1 General

Evaluation of the proposed DW Project structures by DWR, Reclamation, and consultants indicate several structural, environmental, operational and maintenance problems. Management of the proposed 193 small siphons, fish screens and pumping structures needs continuous annual expenditures, which cannot be justified over long-term operations. A consolidated design for intake and outlet facilities is recommended due to the following reasons:

- Avoid or reduce high annual operation and maintenance costs.
- Risk of failure of these structures and impacts on adjacent lands.
- Importance of uninterrupted supplies.
- Impacts of high velocities adjacent to fish screens on fisheries.
- Importance of meeting DWR and Reclamation design standards for the unique nature of soils in the Delta.
- Environmental and recreational use implications around 193 small structures overlying embankments.

Consolidated diversion facilities with different intake types may offer a better solution. Engineered, flat plate screened diversion along river banks have proven to be extremely reliable under a wide variety of flows and conditions, including in the Delta. Examples of facilities using this concept include Contra Costa Water District's new Los Vaqueros Intake Screen (250 cfs), Reclamation District 108's new fish screen (830 cfs), and Glenn Colusa Irrigation District's new fish screen (3,000 cfs). All of these facilities contain fish screens that were developed using the same concept, and have functioned well with minimal maintenance

5.4.2 Selection of Site Locations

To serve the main purpose of diversion and release of stored water, it is important that the integrated facilities be located in areas of minimal controversy. The following factors were considered in the selection of sites for integrated facilities.

- Topography of the area.
- Access to different sources of water.
- Impact of released water quality on other intakes.
- Channel conditions in relation to environmentally sensitive areas.
- State Water Resources Control Board Water Right Permit restrictions.

Topography of various sites was inspected through field reconnaissance and physical locations were decided to meet above constraints. General location maps for the proposed facilities are shown on Figures 4 and 5, Appendix B.

5.4.3 Hydraulic Design Concept Development

For diversions, an integrated facility should be able to draw water from all ranges of river flows. As the reservoir islands are at lower elevations than channels, there are times when water can flow into the reservoir under the action of gravity, or a combination of gravity and pumping. When the channel water level is lower than that in the reservoir, pumping will be required for diversion. Fish screening is necessary to provide protection and avoid fish mortality.

For releases, flow conditions similar to diversion may prevail and reservoir operations may be reversed for gravity flow, a combination of gravity and pumped flows, and pumping-only flow

For varied flow conditions, an integrated structure with fish screens, inlet and outlet transitions, gate controls, pumping units, flow channels and pools to retain sufficient depth for pumping will be required. In addition, a bypass channel will also be required. The following design concepts for these components were developed based on the design criteria given in Section 2.0.

5.4.3.1 Fish Screen Intakes

To meet the 0.2 feet per second approach velocity criterion a wide inlet section is needed. On the other hand, gated structures required for flow control need narrow sections with higher flow velocities. Thus, a long transition structure from a wide section to a narrow section will be necessary.

5.4.3.2 Gate Controls

The gates in an integrated facility serve the following purposes: control the direction of flow during pumped or gravity flow, control the flow velocity during pumped flow, and regulate the flow during gravity flow. These issues are discussed below and require careful consideration during design of the gates.

Control of Flow Direction: The gates will direct the pumped or gravity flow from the river into the reservoir (diversion), or from the reservoir into the river (release).

Control of Flow Velocity at the Gates: For pumped discharge, the velocity at the maximum gate opening should be less than or equal to the velocity at the pump intake.

Flow Regulation: During gravity flow, when the elevation difference between the river and the reservoir water surface is relatively high, the diversion rate or the release rate may exceed those allowed by the applicable permit, unless regulated by the gate opening. The gate opening can be controlled (based on the gate’s discharge vs. head rating) to limit the discharge to the permitted value. Flow regulation by a gate is not required during pumped flow, since the pumps control the flow rates.

5.4.3.3 Low Pool and Bypass Channel

Each integrated facility will require a low pool and a bypass channel. The purpose of providing each of these components is outlined below.

Low pool: The pump intakes are kept submerged to eliminate any air suction and cavitation problems. The low pool provides a pool of water at the pump intake. The pool must be sufficiently deep to cover the entire diameter of the intake pipe. The top of the pipe intake is set below the lowest operational water surface in the river or the reservoir dead storage water level (whichever is lower) to keep the intake pipe submerged under all operational conditions of the river and the reservoir. The pool must be sufficiently long and wide to provide space for four to five intake pipes of the pumping plant and to facilitate periodic maintenance including removal of sediments using mechanical equipment.

Bypass channel: The bypass channel conveys reservoir releases into the river. Reservoir water is released at the upper end of the bypass via conduits connected to the reservoir. Water can flow through the conduits into the bypass either by gravity or by pumping. The discharge through the bypass is isolated from the integrated facility intake area by a sheet pile wall.

The analyzed dimensions of the low pool and the bypass channel are summarized in Table 21 (Appendix B).

5.4.3.4 Pumping Facilities

The pumping facilities should be capable of diverting water from the adjacent river into the reservoir and releasing water back from the reservoir into the river. The criteria for the pump setting was to keep the pump impeller or the turbine blades submerged at all times. Pump flow rating is to be based on the design flow rates (the maximum of diversion and release flow rates) and the total dynamic head (TDH) on the pump. The TDH is the sum of the static head and all the hydraulic head losses in the system. The static head is computed as the difference in water level elevation between the river and the reservoir.

5.4.3.5 Hydraulic Design and Analysis

Layout plans for the proposed integrated facilities for the reservoir islands are shown on Figures 15, 17, 19, 21, 23, 25 and 27, and cross-sectional details of the facilities are shown in Figures 18, 20, 22, 24, 26 and 28, in Appendix B. Further details on the design of these structures are presented in the following sections.

5.4.3.6 Gated Structures Operation

A combination of two gated structures and two valves on the conduits provide coordinated operations with pumping units under various flow conditions as shown in Table 19, Appendix C.

Each gated structure will have three vertical slide gates, arranged between two bridge piers and two abutment walls, as shown in Figure 16, Appendix B. The piers will be extended a short distance beyond the bridge deck for installing trash racks and hoist mechanisms.

The maximum gate opening area was obtained by using the known discharge through a gate, and the fact that the average velocity at the gate must be equal to or less than the velocity at the pump intakes. The velocity at the pump intake is a function of the design flow, the number of intake pipes, and pipe diameter. Once these parameters were computed, the maximum gate opening was calculated for a given gate width, from which the required height of the gate was ascertained.

The computed gate dimensions and related design data are given in Table 20, Appendix C. The specific equations used for computations, along with their sources, are listed in Appendix D.

5.4.3.7 Pumping Plants

Table 22 in Appendix C provides the results of computations of the pump installation capacity for the integrated facilities. A pump efficiency of 87 percent was used in computations. The required number of parallel pump units in an integrated facility was obtained by dividing the maximum flow rate of the facility (the maximum river diversion rate, for example) by the design flow rate of each individual pump unit. An individual pump design flow rate of 500 cfs was assumed for all of the integrated facilities.

The computed pump requirement for a design flow of 1,500 cfs ranged from approximately 2,900 to 3,500 kilowatts for total dynamic head ranging from 20 feet to 24 feet. The total head loss ranged from 2.08 feet to 2.26 feet. The specific equations utilized in computations, along with their sources, are listed in Appendix D.

The bottom elevation of the pump house was set lower than the minimum reservoir water surface elevation (elevation corresponding to about 5 percent of the reservoir storage) or the minimum river water surface elevation, whichever was lower. To ensure that the intake pipe for the pump would remain submerged even when the water surface in the river and the reservoir are at their lowest elevations, the top of the intake pipe was set 2 feet below the previously calculated bottom of pump house elevation. The bottom invert elevation of the intake pipe was obtained by subtracting the diameter of the intake pipe (i.e., 10 feet) from the elevation of the top of the intake pipe. The top elevation of the pump house was set as the new embankment top elevation. The actual elevation may be higher than that of the new embankment if a superstructure for a gantry crane is provided.

5.4.3.8 Fish Screen Intakes

The proposed fish screens will be flat-plate, continuously cleaned design meeting the criteria of the DFG. According to DFG criteria, the gross approach velocity in front of the screen should not exceed 0.2 fps. The screens will be designed to meet the required approach velocity and the length will vary from 1,000 feet to 1,200 feet depending on the capacity of the integrated facility and the depth of the river. The screens will be on the channel side of the integrated facility and they will be placed above the river bottom to help reduce silt buildup. Piles will be used for the foundation and trash racks will be required to protect them.

The inlet gates will be about 550 feet away from the screen. The side embankments will be tapered toward the gates forming a pool of water of varying depth for energy dissipation and control of approach velocity at the screens and gates. During diversions, the expected pump inlet velocity is 6.4 fps and during release the expected velocity at the outlet gate is 3.8 fps. These velocities will assure a flow velocity of 0.2 fps at the screen.

5.4.4 Integrated Facilities Structural Features

5.4.4.1 Basis for Structural Design

The geotechnical information presented in the referenced reports by Harding Lawson Associates (1989) and URS Greiner, Inc. (2000), as well as the findings of a recently concluded subsurface investigation and testing program by Reclamation and DWR, were used in the development of concepts. The upper 25 to 30 feet of soil at the proposed locations of the integrated facilities are composed of soft clays and peat soils, underlain by a layer of medium dense to dense sand having thickness in the order of 50 feet. Due to the high potential for settlement and subsidence of the shallow clay and peat, these soils were not considered suitable for the support of the sensitive structures planned within the integrated facilities (pump house and inlet/outlet gates, for example). Therefore, deep foundations bearing onto the dense sand layer underlying these soils were considered for these structures. Embankments and cut slopes will be designed in accordance to criteria and analysis detailed in Section 4.

5.4.4.2 Structural Design of Components

5.4.4.2.1 Inlet and Outlet Gated Structures

As indicated on the layout plan (Figure 15, Appendix B), a typical integrated facility will have a total of two gated structures to regulate the inflow and outflow of water. Each gated structure will be provided with vertical lift gates and a bridge connecting the road running along the crest of the embankment. The length of each bridge is expected to be about 45 feet and the width about 35 feet. For each gated structure, 3 vertical lift gates are provided. Each gate will consist of a steel panel approximately 12 feet wide and 8 feet high and will be operated from a control room near the bridge.

5.4.4.2.2 Bridge Piers, Abutments and Deck

Each bridge will be supported on two abutments (one on each side) and two interim piers. For the purpose of this review, it has been assumed that each pier will be approximately 35 feet long (i.e., equal to the width of the bridge), 30 feet high and 3 feet wide. In addition, a 2-foot-deep, 45-foot-long and 35-foot-wide bridge deck is considered.

5.4.4.2.3 Foundation

A deep foundation system consisting of square, 2- by 2-foot, driven concrete piles connected at the top by a 3-foot-wide and 2-foot-deep grade beam is considered for each bridge. Six piles, at a spacing of 6 to 8 feet on center, will support each pier. Based on subsurface soil conditions and expected structural loads, a pile length of 35 feet was considered.

5.4.4.2.4 Retaining Walls

Retaining walls are proposed at various locations within each integrated facility. These walls will be constructed along the channel banks of the inlet/outlet-gated structures and will support portions of the control yard area. The profiles of retaining walls will generally be similar at all integrated facilities. The exposed height of the walls will vary from a minimum of 3 feet to a maximum of 35 feet.

Three different cantilever retaining wall sections having exposed heights of 35 feet, 30 feet and 3 feet and supported on shallow foundations have been assumed for this study. Using the exposed heights, the other dimensions of the walls (base width, embedment depth, stem thickness, and base slab thickness) were proportioned in accordance with the guidelines suggested in Das, 1984 and Bowles, 1988, assuming Rankine active earth conditions behind the walls. The retaining wall dimensions assumed in this study are provided in the following table. The actual wall dimensions will be finalized during the design stage.

Dimensions of Proposed Retaining Walls

Height of Wall (ft)	**Width of Base (ft)**	**Stem Thickness (ft)**	**Base-Slab thickness (ft)**	**Depth of Embedment (ft)**
35	20	3.5	3.5	5.5
30	15	3	3	5
3	3	1	1	2

No batter was considered for the stem-walls. A shear key extending to a depth of 3 feet below the bottom of the wall foundation slab was considered for the 35-foot- and 30-foot-high wall sections. While the actual designed depth of the shear keys for individual sections of the walls will vary, 3 feet was considered to be an appropriate average value. A shear key was not considered for the 3-foot-high wall section.

5.4.4.2.5 Control Yard and Pumping Station

As shown in Figures 17, 19, 21 and 23 in Appendix B, each integrated facility will contain a central control yard containing a building housing, a pump house, a maintenance shop, and office/living quarters. The overall area of the control yard will be about 68,000 square feet. Of this, the control building will occupy an area about 12,000 square feet. The remaining area within each control yard will be used as an asphalt-concrete parking area underlain by engineered fill.

The foundation of the control building is expected to be subjected to static loads due to the weight of various equipment, as well as dynamic loads resulting from the vibration of equipment when in operation. A deep foundation system consisting of square,1foot- by 1-foot, driven concrete piles connected at the top by a 3-foot-deep pile cap (constituting the floor of the pump-house) was considered for this study. A pile length of 45 feet was assumed to ensure adequate bearing into the dense sand layer underlying the peat. Three different pile spacings, 10 feet, 15 feet and 20 feet on center were reviewed. A spacing of 10 feet on the center was considered to be appropriate for the expected static and dynamic loads, the assumed pile size, and the pile length.

5.4.4.2.6 Embankments around Integrated Facilities

The embankment crest elevation and sideslopes will be based on the design criteria outlined in Chapter 4. A discussion of the embankment slopes considered in this study for interior the pools and reservoirs is provided below.

Low Pool and Bypass Channel - An embankment with slope of 3:1 and riprap protection on the surface was considered for the interior side of the low pool and bypass channel. Based on review of the subsurface site conditions and the information presented in the referenced reports, the slopes are expected to be stable for long-term static loading. However, the factors of safety during and immediately after construction depend on fill placement time for different stages of construction.

Island/Reservoir Side Embankment - The slope of the embankments on the island/reservoir side utilized in this study was as follows-3:1 slope starting from the crest down to approximately +4 feet elevation, and 10:1 slope thereafter. Based on our review of the subsurface site conditions and the information presented in the referenced reports, the slopes are expected to be stable for long-term static loading. The factor of safety during and immediately after construction could, however, be lower than those defined in the design criteria. The factor of safety for the rapid drawdown case would probably be adequate, provided suitable seepage control and drainage measures are undertaken during design and construction to minimize seepage and to ensure substantial drainage from the embankment soils for the expected rate of reservoir drawdown.

Detailed stability analysis of embankment slopes within the proposed integrated facilities should be undertaken during the next phase of the investigation and the design stage of the project. The analysis should use up-to-date design information and site-specific subsurface information obtained from subsurface exploration and laboratory testing.

5.4.5 Conveyance Facility Design

Results of preliminary engineering design for the facility conveying water from the proposed Victoria Island reservoir to the New Clifton Court Forebay (CCFB) are summarized in this section. The conveyance facility will consist of a pumping plant at the proposed Victoria Island reservoir, siphons, an open channel and a culvert.

5.4.5.1 Siphons

The siphons will convey, partly by gravity and partly by pumping, stored water from Victoria Island into the open channel. The open channel will pass the water through a culvert under a proposed embankment and discharge into the New Clifton Court Intake (presently under study), which will then convey the water into the CCFB (Figure 29, Appendix B).

Four 6-foot diameter siphons were considered, to convey a wide range of flow releases up to the maximum (design) release rate of 2,000 cfs selected from a statistical analysis of the Victoria Island reservoir operational run. The results of analysis show that gravity flow can provide a discharge of up to about 680 cfs with the maximum head differential of 5 feet between the reservoir and the channel water surfaces. Consequently, pumps need to provide a boost in order to obtain the design discharge of 2,000 cfs. The booster pumps will provide additional flows resulting in higher velocities, which will prevent deposition of any suspended sediment in the lower part of the siphon for any prolonged period of time.

5.4.5.2 Inlet and Outlet

Trash racks are provided upstream of the pump/siphon intake to prevent debris from getting into the siphons or the pumps. The trash racks are sized to maintain a through velocity of 2 fps or less to limit head losses. Shutoff gates will be provided just downstream of the trash racks, which can be closed for inspection and maintenance of the pump and siphon units.

The siphon outlet will discharge into a pool of water created by CCFB. The water surface elevations in CCFB can vary between +1.0 feet and –1.0 feet; and the outlet invert is at elevation –8 feet. Consequently, the discharge pool depth varies between 7 and 9 feet at the siphon invert. Since the siphon pipes are 6 feet in diameter, there will be a 1- to- 3-foot depth of water over the top of the siphon pipes. The maximum discharge velocity of the four siphons under the design flow of 2,000 cfs is 17.7 fps. To prevent long-term scour, a riprap blanket 3-foot thick, 70-foot long, 54-foot wide at the beginning and flaring out to 85 feet at the end is provided. The stable riprap rock size (D_{50}) is 19 inches and the maximum rock size (D_{100}) is 36 inches.

5.4.5.3 Conveyance Channel

The open channel has a bottom width of 85 feet, and side slope varying from about 2:1 to 3:1. For the design flow rate of 2,000 cfs, the flow depth is 7 feet and the average velocity, 2.7 fps. This velocity is considered non-scouring and no erosion protection lining is needed. At the end of the channel, six 12.5-foot diameter culverts with a combined area equal to the flow area that corresponds to the design flow of 2,000 cfs in the channel are included. This is to ensure that the culverts do not create any backwater, which would require increased pumping energy and may cause deposition of suspended sediments.

5.4.5.4 Pumping Plant

The pumping plant has 4 pumps, one for each of the four siphons, with a combined flow of 2,000 cfs. With a minimum water surface elevation -8.4 feet and a maximum elevation of +1 feet, the total dynamic head is 27.5 feet and the required installed capacity is 5.3 MW. During gravity flow, only the siphons will be full of water and the flow will bypass the pumps.

CHAPTER 6 – DELTA WETLANDS PROJECT RISK ANALYSIS

6.1 General

In evaluating the proposed project, it is important to understand the risks involved in constructing a water storage project within the Sacramento-San Joaquin Delta. To evaluate risk it is also important to understand the probability of failure of the project. Many factors impact the probability of failure of such a project including foundation conditions, water levels, embankment height, embankment side slopes, embankment composition, slope protection, existing utilities, earthquake loading, and operations.

It is also important to understand the severity and consequences of failure. Risk analysis should cover assessment of impacts of project failure on environment, water quality, reliability of supplies, facilities and infrastructure, economic, health and safety, and land use.

As part of the final DW Project environmental impact statement, the impacts on PG&E's gas transmission lines that cross Bacon Island were discussed and several mitigation options were included. For most of the other factors that impact the probability of failure, the DW Project did not specifically address the issue of risk, but rather attempted to establish engineering criteria to avoid failure. Many of these factors have been discussed in previous sections of the report.

Reclamation and DWR recognize the difficulties and complexities in defining risk in a meaningful way. Reclamation/DWR undertook a review of the previous work performed by others and a simplified risk analysis of the DW Project was done by URS (2001), at the request of Reclamation/DWR. The scope of this risk analysis and major findings by URS are presented in Section 6.3. A separate risk analysis report compiled by URS (2001) is also available.

6.2 Previous Studies

6.2.1 Levee Failure Study

In 1978, Houston and Duncan prepared a report on the probability of failure of levees in the Sacramento-San Joaquin Delta for the Sacramento District of USACE. The report looked at about 100 levee failures in the Delta since the early 1900s. The report separated types of failures into two main categories: overtopping failures and instability failures. It noted that many of the early failures were the result of overtopping. More recently, however, water levels have been better controlled and overtopping failures have decreased considerably. Conversely, in recent years, instability failures have increased. This has largely been due to island floor elevations dropping as a result of subsidence. The lower floor elevations result in increased hydrostatic loading conditions on the levees.

The study ranked the 44 interior Delta islands in terms of probability of overtopping and stability failures. Results of the study indicated Webb Tract and Bacon Island to have the 2nd and 12th highest probabilities of stability failure and 3rd and 14th highest probabilities of overtopping failure, respectively. Because of the proposed improvements for the project, however, these numbers should improve although a new loading condition is being introduced.

6.2.2 CALFED Earthquake Study

Based on historical information, there has been little damage to Delta levees caused by earthquakes (CDWR, 1992). This lack of earthquake-induced levee damage corresponds to the fact that no significant earthquake motion has apparently ever been sustained in the Delta area since the construction of the levee system approximately a century ago. In 2000, CALFED completed a report titled "Seismic Vulnerability of the Sacramento-San Joaquin Delta Levees." Findings from this report indicated that significant numbers of levee failures could occur in the Delta from earthquake shaking. For an earthquake with a 100-year return period, 3 to 10 levee failures were predicted in the Delta on one or more islands. For an earthquake with a 475-year return period (hazard exposure level adopted by DW), 10 to 60 levee failures were predicted. The report also indicated that levees in the Delta were much more susceptible to liquefaction failures than non-liquefaction failures by a factor of about 10 to 1.

6.3 Risk Analysis By URS

6.3.1 General

The object of the risk analysis was to assess the probability and consequences of failure of the embankments proposed for the DW Project. Failure was defined as an uncontrolled release of water from the reservoir or into the reservoir from adjacent river channels, either from a failure of the proposed embankment or appurtenant structures. Risk was quantified in terms of exceedance and consequences of failure scenarios were described qualitatively.

Risk of failure of the DW Project was evaluated for operational, flooding and seismic events. A brief description of each type of analysis is presented here. Details are available in URS risk analysis report (2001).

6.3.2 Operational Risk

The operational risk analysis involved evaluation of embankment seepage and slope stability, potential failure modes and their associated probabilities under operational conditions.

Under slope stability, sensitivity analyses were conducted regarding the angle of the slopes on the slough side of the embankments, the depth of adjacent sloughs, and possible variability in the average strength of the weakest material, peat. This was carried out to quantify how the uncertainty or variability in such parameters influences the overall risk. URS used the results of their earlier study (2000) to briefly review the risk associated with seepage and under seepage, and the potential for piping that could affect adjacent islands.

A review of previous studies, and additional analyses were conducted to assess the operational risk of the constructed reservoirs at Bacon Island and Webb Tract. The review and analyses indicated that, under long-term normal operation, the probability of slope failures on the slough-side of the embankments would increase.

The potential for seepage-induced piping and erosion could be high if high water heads are allowed to build behind the embankments without seepage-control measures. The DW Project proposes constructing interceptor wells to control seepage. With the proposed interceptor wells and proper operation and maintenance, this risk of seepage-related damage will be substantially reduced. However, in case of power failures, seismic event and flooding failure, the interceptor well system may be damaged or may be out of operation, the consequences of which may be severe.

6.3.3 Flooding Risk

Probability of flood-overtopping failure of the proposed storage reservoir islands at Webb Tract and Bacon Island was evaluated for 100-year river flood stages for the Delta Wetlands Project. Flood-overtopping risks at Webb Tract and Bacon Island reservoirs were evaluated based on the flood stages and wind-wave characteristics estimated for the Sacramento-San Joaquin Delta region. The probability of embankment failure due to flood events considered only the potential failure due to overtopping.

A stage-frequency analysis for the design floods for the 50-year and 300-year return periods was performed and probabilities of failure for these events were also estimated for sensitivity purposes.

The greatest flood-overtopping depth is expected to occur at Webb Tract from Station 70+00 to 220+00, the section opposite to Frank's Tract. Other sections at Webb Tract are also expected to overtop during the 100-year flood. Results show that 8 embankment sections at Bacon Island are expected to overtop during the 100-year flood. For all sections that overtop, the probability of overtopping-failure during the 100-year floods were estimated to be 39 percent in a 50-year life cycle. For all floods, (50-year, 100-year, and 300-year) the probability of overtopping (when flood stage and wave run-up is higher than the embankment crest) was equivalent to the probability of the occurrence of the flood.

6.3.4. Seismic Risk

URS evaluated the material properties of existing levees and foundations for dynamic analyses. The CALFED seismic vulnerability study (2000) was used to develop a range of up to 4 earthquake events and their respective probabilities of occurrence. Based on the work already developed by the seismic vulnerability sub-team of CALFED, ground motions were developed at the site relating peak ground accelerations for stiff-soil sites to probability of exceedance. URS used the relationships between stiff soil accelerations, average peak acceleration and levee displacements from the CALFED studies, together with the estimated yield acceleration to estimate permanent deformations induced by earthquake shaking. Seismic vulnerabilities of both reservoir islands were determined by identifying the probabilities of embankment failures based on the pre-defined failure criteria and the results of the CALFED studies. These probabilities of failures are considered to be underestimates, since they were developed for the existing levee system and do not account for the reservoir loading.

Results of the seismic vulnerability study indicate that there is about 5.5 percent chance in 50 years that Bacon Island embankments will fail due to earthquakes. The corresponding probability of failure for the Webb Tract embankment is about 8.5 percent in 50 years. Foundation soils for the embankments at Webb Tract have higher susceptibility to liquefaction than those at Bacon Island. The probability of the DW Project embankment failure due to seismic loading was determined to be higher than the Clifton Court Forebay.

6.3.5 Consequences of Embankment Failure

Overall, the impact rating to the infrastructure, economy, land use, and health and safety resources are generally medium to low because of the relatively non-urbanized affected area that have lower asset values compared to more urbanized and developed areas. However, impacts due to failure on the Discovery Bay housing development may have higher damage potential. High consequences would be observed in water quality and biological resource categories. For water quality, the potential salt-water migration to the discharge pumps during an inward embankment breach could affect water users who depend on Delta water. For biological resources, some fish species may suffer from entrainment into the reservoir during an inward breach. Fish could be trapped inside the reservoir once the higher slough water starts to recede. The magnitude of impact may vary depending on the fish species and life stage present during migration periods.

CHAPTER 7 – QUANTITIES AND COST ESTIMATES

7.1 Impacted Adjacent Structures And Relocations

The reservoir islands are agricultural areas with no urban development. Processing plants and other installations associated with agricultural operations include warehouses, canneries and structures like barns and sheds, which will need relocation before reservoir filling. Other relocations will consist of public and private infrastructure, the most significant being the PG&E gas pipeline and Highway 4.

PG&E proposed realignment of the gas pipeline at a cost of $22 million. The pre-feasibility report cost of $9.2 million was used in this study.

7.2 Quantity Estimates

Embankment fill quantities were estimated based on cross sections of existing levees from the "Sacramento-San Joaquin Delta Levee Rehabilitation Study", CALFED, September 1998[4]. Earthwork items included toe berm, crown fill, slope protection, pumping plant excavation and fill and road base.

Staged construction will be required for modifying the embankments due to the organic materials. As discussed in Section 4.5, the embankment quantities were increased by a factor of 2.25 to compensate for settlement of the soft foundation materials.

Installation of an interceptor well system with 554 wells on Bacon Island and 219 wells on Webb Tract is included in the cost estimate. Well depth of 80 feet, with the bottom 40 feet in sand with a six-inch diameter perforated PVC casing, were considered.

Embankments, ditches, and channels required on the habitat islands were assumed the same as proposed by DW. No analysis or designs were performed at this time to refine the estimates. Future designs should determine how much excavation is required, in addition to excavations for borrow operations.

Estimates included for comparison in this report are for the re-engineered DW Project and the two alternatives that involve Victoria Island. Slough-side rock riprap quantities for 3:1 slope modifications were estimated by using cross sectional data compiled by Murray, Burns and Kienlen Engineers, based on 1996 soundings by Kjeldsen, Sinnock and Nendeck, Inc. The locations of these soundings are shown on Figure 12.

Estimates for the proposed DW Project by Delta Wetlands Properties is not included, as it does not meet the Reclamation/DWR standards for public ownership and can therefore not be compared with the alternatives presented in this study.

7.3 Construction Access and Constructability

7.3.1 Access

There are public roads to Bacon Island, but not to Webb Tract. The route to Bacon Island includes a drawbridge between Lower Jones Tract and Bacon Island. It is assumed that most of the fuel, equipment, materials, supplies and work force will be brought to the project islands by barge or boat. Most of the fuel, equipment and supplies will come by barge from Antioch, involving about a 4- to 5-hour one-way trip. Most of the slope protection materials would come by barge from San Rafael, involving an approximately 12-hour one-way trip. Neither time estimate includes time for loading or unloading. For unloading, docks and ramps will be required at the project islands

During the next phase of study, the feasibility of constructing an access road to Webb Tract should be reviewed. Additionally, necessary modifications to the access road to Bacon Island should be investigated.

7.3.2 Equipment

The most efficient means to construct the proposed embankment modification is to use lightweight, rubber-tired equipment. Due to the instability of the foundation conditions, small scrapers with capacities of about 13 cubic yards will need to be used to avoid soil pumping on haul roads and in the embankment areas. Production may be limited by the poor foundation conditions.

7.3.3 Mobilization/Demobilization

It is assumed that earthwork will be carried out between the normal construction season for this area from May through October. Wet weather will make foundation conditions difficult for construction equipment and may limit earthwork to six months out of each year.

7.3.4 Other Construction Issues

The project islands contain soils that can be used as borrow materials, however, the volume of such soils available only on the surface of the islands may not be sufficient. The borrow operation may therefore require excavating the overlying peat soils to expose suitable subsurface materials. Groundwater is at or slightly below the existing ground surface, therefore, dewatering and/or unwatering may be required. In addition, excavated borrow materials may be saturated, and thus require spreading and drying before placement as fill. As a result of this work, the unit cost of fill material may be higher than usual. Cracking will occur during construction as the soft soils settle under the load of the newly placed embankment fill. Consequently, additional work will be required to rework or fill-up cracked areas to ensure a watertight structure.

Environmental permits related to project construction will be required, the costs of which are included in the estimates.

7.4 Operation and Maintenance

Operation and maintenance activities for the reservoir islands will include the following:

- Inspection and maintenance of the perimeter embankments, including vegetation control, damage repair, fill placement to raise the embankments, and placement of rock revetment as needed.
- Maintenance of the inner embankments for shallow-water management and management of reservoir bottoms.
- Operation of onsite pumps and siphons during water diversions and discharges.
- Maintenance and monitoring of the integrated facilities and fish screens.
- Operation and maintenance of seepage interceptor system.

Operation and maintenance activities for the habitat islands will include the following:

- Operation and routine maintenance of siphon and pump units.
- Management of habitat areas, including control of undesirable plant species, maintenance or modification of inner embankments and water circulation in ditches, canals, open water and shallow flooded habitats to facilitate flooding and drainage.
- Fish screen maintenance and monitoring during water diversions for habitat maintenance.
- Wildlife and habitat monitoring.
- Perimeter embankment inspection and maintenance.
- Maintenance and operation of recreational facilities.
- Operation and maintenance of seepage interceptor system.

Detailed cost estimates for operation and maintenance of the different storage alternatives are provided in Table 23, Appendix C.

7.5 Project Cost Estimate

It was decided by Reclamation and DWR that a cost range based on uncertainties be included in this evaluation for the Re-engineered DW Project. These uncertainties, which include the extent of design based on level of protection, site conditions and construction methods, unit cost variations, and changes in hydrology, are explained under in Section 7.5.3 (Sensitivity Analysis).

A low range cost summary of the three alternatives is presented in the following table, and Tables 24A, 25 and 26 in Appendix C provide detailed cost breakdowns. These costs are based on modifications to the existing slopes on the slough-side and providing additional embankment sections on the reservoir side with variable slopes of 3:1 down to plus 4 above MSL and 10:1 to the toe of the embankment. It is assumed that an annual embankment maintenance program will be developed under the DFG AB 360 Levee Maintenance Program. The modified slough-side 3:1 slope will be maintained with annual maintenance, and the cost of which is included in the operation and maintenance costs presented in Table 23 (Appendix C).

Detailed estimates of various components of the integrated facilities were obtained from layout plans and cross sections shown in Figures 17 through 31 (Appendix B).

Although preliminary cost evaluations were done for the DW proposal, these are not included in this report. The DW Project does not meet the design standards imposed on the Re-engineered and other alternatives and showing costs for an under designed project is not a fair comparison.

IN-DELTA STORAGE PRELIMINARY PROJECT COST ESTIMATE

Cost Item	Project Alternatives (Low Range Costs)		
	Re-engineered Delta Wetlands	**Bacon/Victoria with Connection to Clifton Court**	**Webb/Victoria with Connection to Clifton Court**
Integrated Facilities	$ 104,733,000	$ 95,855,000	$ 96,698,000
Fish Screens	$ 60,000,000	$ 60,000,000	$ 60,000,000
Seepage Control System	$ 10,634,000	$ 5,000,000	$ 5,000,000
Land Acquisition	$ 59,814,000	$ 64,998,000	$ 64,770,000
Facilities(Conveyance to CCF)		$ 36,910,000	$ 36,910,000
Island Embankments	$ 144,559,000	$ 152,523,000	$ 152,154,000
Demolition, Cleanup and Miscellaneous	$ 8,100,000	$ 8,100,000	$ 8,100,000
Permits	$ 300,000	$ 300,000	$ 300,000
New Utilities / Relocations	$ 12,380,000	$ 63,134,000	$ 53,934,000
Interior Work	$ 2,400,000	$ 2,400,000	$ 2,400,000
Mitigation	$ 21,000,000	$ 21,000,000	$ 21,000,000
Subtotal	**$ 423,920,000**	**$ 510,220,000**	**$ 501,266,000**
Mobilization (5%)	$ 21,196,000	$ 25,511,000	$ 25,063,000
Contingencies / Unlisted Items (20%)	$ 84,784,000	$ 102,044,000	$ 100,253,000
Construction Cost Subtotal	$ 529,900,000	$ 637,775,000	$ 626,582,000
Engineering, Legal and Admin. (25%)	$ 132,475,000	$ 159,444,000	$ 156,646,000
TOTAL PROJECT COST	**$ 662,375,000**	**$ 797,219,000**	**$ 783,228,000**

Note: Costs rounded off to nearest $1000

Unit costs are based on Reclamation and DWR experience of construction in the Delta and costs used in the previous CALFED/Reclamation/DWR pre-feasibility studies. Previous studies information was updated with the Engineering News Record Index to bring it to the present level cost. Detailed cost estimates for items shown in Table 10 are given in Appendix C of the *In-Delta Storage Draft Report on Engineering Investigations,* April 2002. Sherman Island Reservoir preliminary costs were also estimated for information and comparison purposes only and are approximately $837,582,000 for a storage capacity of 179 TAF.

7.5.1 Contingency and Engineering Costs

An initial mobilization cost of 5 percent was assumed. Project contingency costs were assumed to be equal to 20 percent of the base construction estimates. The engineering, legal and administrative costs were assumed to be equal to 25 percent of the subtotal of the base construction estimates plus contingencies. This cost component would account for project planning as well as engineering design (conceptual through final) and construction management. Lastly, legal and administrative costs associated with land acquisition, construction contracts and infrastructure relocation are considered in this component. These assumptions are typical for projects of this magnitude.

7.5.2 Annual Maintenance and Operation Costs

Annual maintenance and operation costs for the various alternatives are provided in Table 23. These costs include the items listed in Section 7.4, and were estimated based primarily on existing information, as summarized below. Wherever applicable, engineering indices were used to update existing cost information.

- Embankment maintenance expenses were estimated from information provided by Central District.
- Maintenance costs for integrated facilities and fish screens include the cost of one person-year for each facility (total of four), and yearly equipment replacement costs estimated at 5 percent of the initial equipment costs for the four integrated facilities, over a 50-year project life.
- Costs related to pump operations and maintenance include annual cost of power and maintenance. Total annual pumping amounts were used from modeling operation studies.
- Maintenance costs for the seepage control system were estimated as 5 percent of the cost of installing of the seepage control system.
- Costs for habitat islands monitoring, operations and maintenance were estimated based on maintenance costs at Vic Fazio Wildlife Area and Stone Lakes National Wildlife Refuge. The estimate includes the costs of invasive weed control on the habitat islands.
- Cost estimates for invasive weed control on the reservoir islands were based on the costs of DWR's aquatic weed control at Clifton Court Forebay.
- The estimated cost for fisheries mitigation and monitoring was based on mitigation and monitoring required for Delta Wetlands in USFWS and NMFS biological opinions and DFG regulations.
- The estimated cost for mitigating cultural resources were provfided by the Environmental Services office, and represents the cost for yearly cultural resource compliance with Historic Properties Management Plan for the life of the project.
- Expenses for maintaining facilities associated with recreational use of the four islands were obtained from on a November 30, 2001 Recreational Opportunities Study carried out by CH2M HILL.
- Property taxes for the islands were obtained from County records.

7.5.3 Sensitivity Analysis

The proposed project costs may vary depending on a number of factors such as the extent of design based on level of protection, site conditions and construction methods, and changes in hydrology. A sensitivity analysis was done to determine the potential change in low range costs due to variations in the Re-engineered Delta Wetlands Project design, impact on unit costs due to site conditions and potential problems during construction, and seasonal variations in design inflows due to climate change.

7.5.3.1 Cost Variations based on Design

The following factors were considered.

Embankment Slopes: Changes in the slough-side embankment slopes from the existing slopes to achieve higher factors of safety can cause the following variation in cost.

- Embankment costs will include excavation of slough-side slopes under water and above to flatten slough-side to a 4:1 slope.
- New embankment with 4:1 slough-side slope will be starting at a location inside the existing embankment due to embankment crest starting at a point anywhere from 15 to 30 feet inside towards the island. It will practically be a new embankment and costs will be much higher than the costs shown in Table 10.
- Mitigation costs for shallow water habitat and in stream botanical resources. These costs alone can vary from $100 million to $200 million and depending on the potential of damage to fisheries and other resources in the area can also end up in a Jeopardy Decision.

Piping Protection: A sand filter or some other measure like slurry wall may be required for piping protection. Additional work in this area is required to minimize the possibility of piping failures due to settlement-related cracking. The Independent Board of Consultants recommended against installation of such a filter during the construction or initial stages of the project because it stands a higher risk of construction and post-construction damage. Costs of installing a sand filter were included for both low and high range of costs as a placeholder for piping protection. However, these costs may be different if a slurry wall is selected as a measure for piping protection.

The Independent Board of Consultants recommended that physical design should be integrated with the level of protection. A detailed risk analysis is required to determine a reasonable level of protection. Further design evaluations are recommended for the final design.

7.5.3.2 Cost Variations Based on Site Conditions and Construction Methods

Information on construction methods and unit costs is available from the past 10 years of construction experience in the Delta and discussions with DW. The Reclamation Districts are maintaining Delta levees and DWR pays 75% of the maintenance costs. However, during preliminary cost assessment process, it became evident that differences of opinion exist on the unit costs. The following variations in cost were identified.

- The present levee maintenance indicates $5 to $6 per cubic yard of embankment fill. However, if in future borrow areas are identified to be under the peat layer and removal of organic materials and dewatering is required, this cost may vary from $8 to $15 per cubic yard. A cost of $8 per cubic yard was used for low range estimate and $15 per cubic yard was used for high range estimate.
- Riprap costs in the Delta vary from $18 to $20 per ton based on recent construction costs. However, unit costs may be higher depending on site access problems and other factors such as delivery, placement, overhead and profit, and union wage rates. Considering these factors, unit costs varying from $26 per ton to $39 per ton were used for rock riprap.

A higher range of cost for the Re-engineered DW Project, considering the variations included in the sensitivity analysis is shown in the following table. Details of these costs are presented in Table 24B (Appendix C).

RE-ENGINEERED DELTA WETLANDS PROJECT COST VARIATION DUE TO UNCERTAINTIES

Cost Item	Re-engineered Delta Wetlands (High Range)
Integrated Facilities	$ 132,864,000
Fish Screens	$ 60,000,000
Seepage Control Systems	$ 10,634,000
Land Acquisition	$ 59,814,000
Island Embankments	$ 276,505,000
Demolition, Cleanup and Miscellaneous	$ 8,100,000
Permits	$ 300,000
New Utilities / Relocations	$ 12,380,000
Interior Work	$ 2,400,000
Mitigation	$ 121,000,000
Subtotal	**$ 683,997,000**
Mobilization (5%)	$ 34,200,000
Contingencies / Unlisted Items (20%)	$ 136,799,000
Construction Cost Subtotal	$ 854,996,000
Engineering, Legal and Administrative (25%)	$ 213,749,000
TOTAL PROJECT COST	**$ 1,068,745,000**

Note: Costs rounded off to the nearest $1000.

7.5.3.3 Climate Changes

Global warming and a rise in sea level may add additional constraints for the embankments as designed, by increasing the potential for failure due to overtopping. The embankments will need to be raised to meet water level changes due to this potential change in climate, and associated project costs may be much higher than estimated. The following evaluations were done in consideration of possible climate change.

Project Failure due to Overtopping: Instead of a 1:100-year flood presently considered for embankment design, a higher magnitude 1:300-year flood may become the controlling design criteria for embankments and structures. Such a flood may require that embankment heights be increased by 0.5 to 1 foot. Risk analyses indicate a 16 percent probability of overtopping of the DW Project during the assumed 50-year life of the project.

Increase in Project Quantities and Costs: Increased height to protect against climate change will require that embankments and structures be built to withstand failures. Two increases in height were assumed for sensitivity analyses, 0.5 and 1 foot.

Figure 32, Appendix B shows the variation of costs due to potential change in climate. If the project is designed and built for the presently accepted design criteria of a 1:100-year flood, this additional cost may be spread over the next 50 years of the project life in the form of increased annual maintenance and operation costs to avoid overtopping and failures. As the base cost of $662.4 million may increase to $1.1 billion due to changes in design, unit costs, site conditions and construction methods, costs shown in Figure 32 for 0.5 and 1.0 foot levels may increase by 10 to 15 percent due to these factors.

CHAPTER 8 – PROJECT EVALUATION

8.1 General

To assess the engineering feasibility of the DW Project, the following evaluations were undertaken:

- Delta Wetlands Embankment Design as described in the 2001 EIS/EIR (Prepared by Jones & Stokes for Delta Wetlands Properties).
- Fish Screens, Siphons and Pumping Facilities Evaluations
- A simplified risk analyses of Delta Wetlands Project
- Alternative Design Proposals
- Quantity and Cost Estimation

The embankment and inlet/outlet structural designs proposed by the DW Project for Webb Tract and Bacon Island do not meet DWR and Reclamation design standards and are therefore inadequate for public ownership by these two agencies. A review of the results of geotechnical investigations, engineering design and risk analyses led to the following findings, conclusions, and recommendations.

8.2 DW Embankment Design

8.2.1 Findings and Conclusions

8.2.1.1 General Embankment Design

- The DW Project-proposed crest elevations (approximately 9 feet MSL) would meet the height criteria for the reservoir side only. The crest elevation required to prevent overtopping from wave action on the design river flood elevation would not be met along the entire perimeter. For Webb Tract, 46,000 linear feet of embankment (68 percent) would be overtopped and 16,000 linear feet (26 percent) would come close to being overtopped during the 100-year flood. For Bacon Island, 26,800 linear feet of embankment (36 percent) would be overtopped and 32,400 linear feet (43 percent) would come close to being overtopped during the 100-year flood.
- The new embankments proposed by the DW Project would not alter the river/slough slope. Two configurations for the reservoir/island side slopes were proposed: a constant 5:1 slope, and a dual slope of 3:1 from the crest down to elevation -3 then a toe berm with approximately 10:1 slopes. A 2001 EIR/EIS, prepared by Jones & Stokes for Delta Wetlands Properties, indicated that more detailed studies would be done during final design to refine what slopes are needed in specific areas. The proposed river/slough side slopes are technically not acceptable as discussed in the static stability analysis sections.
- The DW Project indicated that the modification would be done with one type of material placed directly on the existing structure. The material would come from natural sand deposits on the islands and from dredge spoil sites. There is a sufficient quantity of soil materials for construction on the islands. Riprap for slope protection would come from a commercial source.
- As discussed in the settlement section, the embankments would be subject to cracking from differential settlement and there is a potential for piping. To minimize the potential for a piping failure, DW proposed placing sand against the inside of the existing levees. No specific information regarding the type of material, thickness, or basis for where it would be placed was provided.

8.2.1.2 Static Slope Stability Analysis

- The method of analysis and assumptions used in the DW Project analyses are appropriate.
- There is no clear definition of the design criteria used in the DW Project. The 1995 Draft EIR/EIS states “Levee improvements would be designed to meet or exceed state-recommended criteria for levees outlined in DWR Bulletin 192-82.” The DWR bulletin addresses levees only and not structures

used for water storage. Also, the bulletin provides no design criteria but identifies a typical levee section and states that specific designs are required for each site. Embankment design criteria similar to small dams should be used instead of levee criteria. The embankments should not be considered levees because of the longer duration of water stored against them and the greater consequences than existing levees in the area if failure were to occur.

- End-of-construction factors of safety for un-staged construction are unacceptable and staged construction over several years will be required to construct the new embankments. A five-year construction period is assumed, but further analysis will be required during final design to refine this estimate.
- The long-term loading condition analyses indicate that the existing river/slough side slopes, in general, do not meet the design criteria (Section 4.2.2.3) and in most areas are considered unacceptable. This is of concern because this is a new loading condition due to the reservoir and the consequences from a failure are different than if an existing levee were to fail. In an telephone conversation with Reclamation and DWR on March 14, 2002, DW recognized that improvements on some areas of the river/slough side will be needed based on the modifications now being considered for the existing levees. The specific areas requiring improvement will be determined during final design.
- The long-term loading analysis indicated the proposed slopes on the island side are acceptable. Analysis that is more complete should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.
- The study recognized that some sections had inadequate factors of safety for the sudden drawdown condition and revisions to the proposed configuration would be required in these areas.

8.2.1.3 Seepage

- Seepage analyses indicate water levels under adjacent islands will rise due to the project.
- The potential for seepage-induced piping and erosion could be high if high water heads are allowed to build up, without seepage control measures.
- The proposed DW Project provides for the construction of interceptor wells to control adverse seepage conditions. This system will have a high operation and maintenance cost that needs to be accounted for in the overall cost of this project.
- The interceptor well system is prone to failures due to geotechnical conditions, seismic events, and local power interruptions or major power failures. For example, power loss or grid failures may last from days to weeks, or even months, in some major historic earthquakes. While backup such as diesel operated pumps is contemplated for the well system, local or distant large earthquakes could cause extended power failures, or even prevent or limit access to the backup pumps for a significant duration of time. Interruptions in the operation of the system could cause localized flooding of adjacent islands.

8.2.1.4 Settlement

- Large settlements of the embankments will occur during initial construction. While the rate and degree of settlement will reduce with the passage of time, the phenomenon will continue throughout the life of the project. The 2001 EIS/EIR indicates that settlement during construction will be compensated by the placing of additional fill. The embankments will have to be raised in the future to maintain the current height.
- Subsidence of the reservoir islands will decrease because of the change in use of the area.
- Severe cracking of the embankments could occur during construction and over the life of the project. The 2001 EIS/EIR indicated crack repair and defensive design measures would need to be determined during final design.

8.2.1.5 Seismic

- The DW Project predicted earthquake-induced deformations of 1.5 to 3.5 feet for the island side slopes and 3 to 4 feet for the river/slough side slopes. The predicted displacements would result in severe cracking and possible failure from erosion through cracks or an overtopping failure due to slumping and loss of freeboard.

- The DW Project concluded some liquefaction and failure of the embankments may occur but it would be no worse than what now exists. This conclusion is not consistent with studies done by others and the resulting impacts could be greater than current conditions due to reservoir storage.

8.2.2 Recommendations

The proposed embankment designs for Webb Tract and Bacon Island do not meet the Reclamation/DWR design criteria for the project and is therefore considered inadequate for public ownership by these two agencies. The two major concerns are the potential for embankment sliding into the river/sloughs and being overtopped by wave action during the design flood event. Additional data acquisition and analyses must be done during final design to refine required cross sections along differing reaches of the embankment to prevent or reduce this potential.

8.3 DW Project Fish Screens, Siphons and Pumping Stations

8.3.1 Findings and Conclusions

8.3.1.1 Design and Environmental

- The proposed fish screen design does not comply with current regulations, such as cleaning requirements, screen mesh and perforated plate opening size, and screen area.
- Pipe velocities at maximum flows through the proposed 36-inch diameter intakes are well above those generally used in pipelines when head loss, interior pipe erosion, and pipe stability are of concern. A more common velocity limitation would be around 10 fps. A larger diameter pipe or two pipes could be considered to reduce velocities, lower energy costs on the in-line pump station, and reduce scour at the pipe outfall. Considering the number of units proposed, modeling of the screen units would be necessary.
- The proposed project will have the direct impacts of impingement, entrainment, and localized predation losses due to the facility.
- There will be long-term environmental impacts related to maintenance activities, access improvements, visual impairments, noise, recreation, and project lighting.
- The embankment slumping, deformations, and lateral spreading will cause over stressing of the siphons and pumps. The truss frame supporting the pumps on the reservoir side may experience strong ground shaking and deformation.
- If overtopping of the embankments during the design flood takes place at the proposed location of pumps and siphons, severe damage to these facilities may occur

8.3.1.2 Operation and Maintenance

The proposed design will be difficult and expensive to operate and maintain for the following reasons:

- No automatic screen cleaning system.
- Poorly retrievable system, even when raised, it will still be subject to corrosion, and poor access for inspection.
- No monitoring system.
- Poor access to the screen.
- Dissimilar metals on pipe and screen.
- Woven wire screen (stainless steel is good but not resistant to bio fouling).
- Appears to be structurally inadequate as described above.

8.3.2 Recommendations

Designing major fish-screen intakes for the DW Project must include considerations related to fisheries protection; structural, hydraulic, and geotechnical design; and operations and maintenance. Successful fish screens are dependent on all of these factors. The proposed design does not appear to include these considerations and is therefore a deficient or a risky design.

8.4 Risk Analysis

8.4.1 Findings and Conclusions

- The operational risk of embankment failure will be small compared to seismic and flood risks. The highest potential risk is expected to be due to overtopping during a flood.
- The probability of embankment failure during construction with release of water from the adjacent slough into the reservoir area would be significant (greater than 50 percent), if construction proceeds too rapidly or without staging.
- For the long-term loading condition, the probability of slope failures on the slough-side of the embankment will be increased. The study assigned the risk of these failures as marginal to unacceptable.
- The study estimated that there is about 5.5 percent chance in 50 years (0.11 percent annual probability) that the Bacon Island embankments will fail during future earthquakes. The corresponding failure probability for the Webb Tract embankments is about 8.5 percent in 50 years (0.18 percent annual probability).
- For all sections that overtop, probabilities of overtopping failure during a 100-year flood were estimated to be 39 percent for a selected project life of 50 years (0.01 annual probability).
- Overall, the impact rating to the infrastructure, economy, land use, and health and safety resources are generally medium to low because of the relatively rural area that has lower asset values compared with more urbanized and developed areas.
- As a result of failure of the Bacon Island embankment due to flood overtopping or seismic failure. A high potential of damage exists for the Discovery Bay housing development.
- High consequences would be observed in water quality and interruption of water supplies. For water quality, the potential salt-water migration to the discharge pumps during an inward embankment breach could affect water users who are dependent on Delta water.
- An inward breach of the reservoir could entrain some fish species. Fish could be trapped inside the reservoir once the higher slough water starts to recede. The magnitude of impact may vary depending on the fish species and life stage present during migration periods.

8.4.2 Recommendations

Solutions should be developed to enhance the reliability of the project to meet the design criteria. As part of this process, focused field investigation and laboratory tests should be developed to address the specific requirements for the desired level of project reliability.

8.5 Re-Engineered Design Proposals

8.5.1 Findings and Conclusions

8.5.1.1 Embankment Design

- The required crest elevation, based on a 100-year flood, varies from 9.6 feet to 15.1 feet with an average of 10.2 feet on Bacon Island and 10.9 feet on Webb Tract. A variable crest elevation should be considered during final design to minimize required new fill construction.
- A crest width of a minimum of 35 feet should be provided to accommodate traffic and future crest increases.

- Slope protection should be provided on both sides of the embankment to protect from erosion caused by the river and the reservoir.
- The existing river/slough side slopes have areas which do not met any design criteria for stability. Additional data gathering and analysis are needed to specifically identify areas that are unstable and require modifications. Additional analysis and design are required to determine what modifications could provide the needed stability and are environmentally acceptable. At this level of study, to add the cost for this modification it was assumed the slopes would be cut back to a 4:1 slope above elevation 0. As an alternative, the cost for 3:1 slope modification of the existing slopes was also included in this study.
- At this level of study, an island/reservoir-side slope of 3:1 down to elevation +4 and a slope of 10:1 below that elevation are recommended. Analysis that is more complete should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.
- To account for the cost of providing some means of protection from a piping failure that could occur from cracking due to settlement, the possibility of constructing an engineered filter zone was considered as part of this study. Other alternatives for providing protection against such failure should be reviewed during final design.
- There is sufficient quantity of materials within the islands to meet the volume of either alternative (DW Project and re-engineered alternative). The material in the borrow areas is expected to be saturated and will require drying.
- The design and cost should include provisions for maintenance work for maintaining an acceptable crest elevation resulting from settlement.
- Factors of safety for the Clifton Court Forebay embankments are higher than those computed for the Bacon Island and Webb Tract due to different foundation conditions. Therefore, steeper slopes could be used for embankments on Victoria Island.

8.5.1.2 Structures

- Consolidated diversion facilities looking at different intake types may offer a better solution.
- Engineered, flat plate screened diversion along river banks have proven to be extremely reliable under a variety of flows and conditions, including the Delta.
- Future design efforts should consider the level of operations and maintenance that will be required for the intake facility verses only looking at initial capital costs. Consolidated facilities using flat plate screen technologies appear to show promise for this application.

8.5.2 Recommendations

- Additional data acquisition and analyses should be done during final design to refine required cross sections along differing reaches of the perimeter embankment.
- Analysis that is more complete should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.
- The need and type of the dewatering system should be evaluated during final design. It is also recommend that at the time of initial construction, material be excavated and stockpiled for future use in emergency repairs for sliding, cracking, or slumping due to an earthquake.
- Alternative seepage control designs should be considered during final design.

8.6 Climate Change Impact

8.6.1 Findings and Conclusions

Global warming and sea level rise may add additional constraints for the embankment as designed. Embankments would need to be raised to meet water level changes due to potential climate change.

8.6.2 Recommendations

It is recommended that further research be undertaken to assess climate change impacts on the project.

8.7 Quantity and Cost Estimate

- Although a preliminary assessment of costs was done for the DW Project, these costs are not included in this report, as the proposed project does not meet the Reclamation/DWR design standards for public ownership.
- Reclamation/DWR estimated appraisal level quantities and cost estimates for the re-engineered design and the two alternatives involving Victoria Island.
- Depending on the design considerations, site conditions and construction methods, the estimated costs for the Re-Engineered Project with Webb and Bacon Islands range from $662 million to $1.1 billion. Operation and maintenance costs are estimated to be $8.3 million annually.
- The estimated cost for the re-engineered project with Bacon and Victoria Islands is $797 million. The operation and maintenance costs are estimated to be $8.4 million annually. This project alternative provides a direct connection to Clifton Court Forebay.
- The estimated cost for the re-engineered project with Webb and Victoria Islands is $783 million. Operation and maintenance costs are estimated to be $8.3 million. This alternative also provides a direct connection to Clifton Court Forebay.

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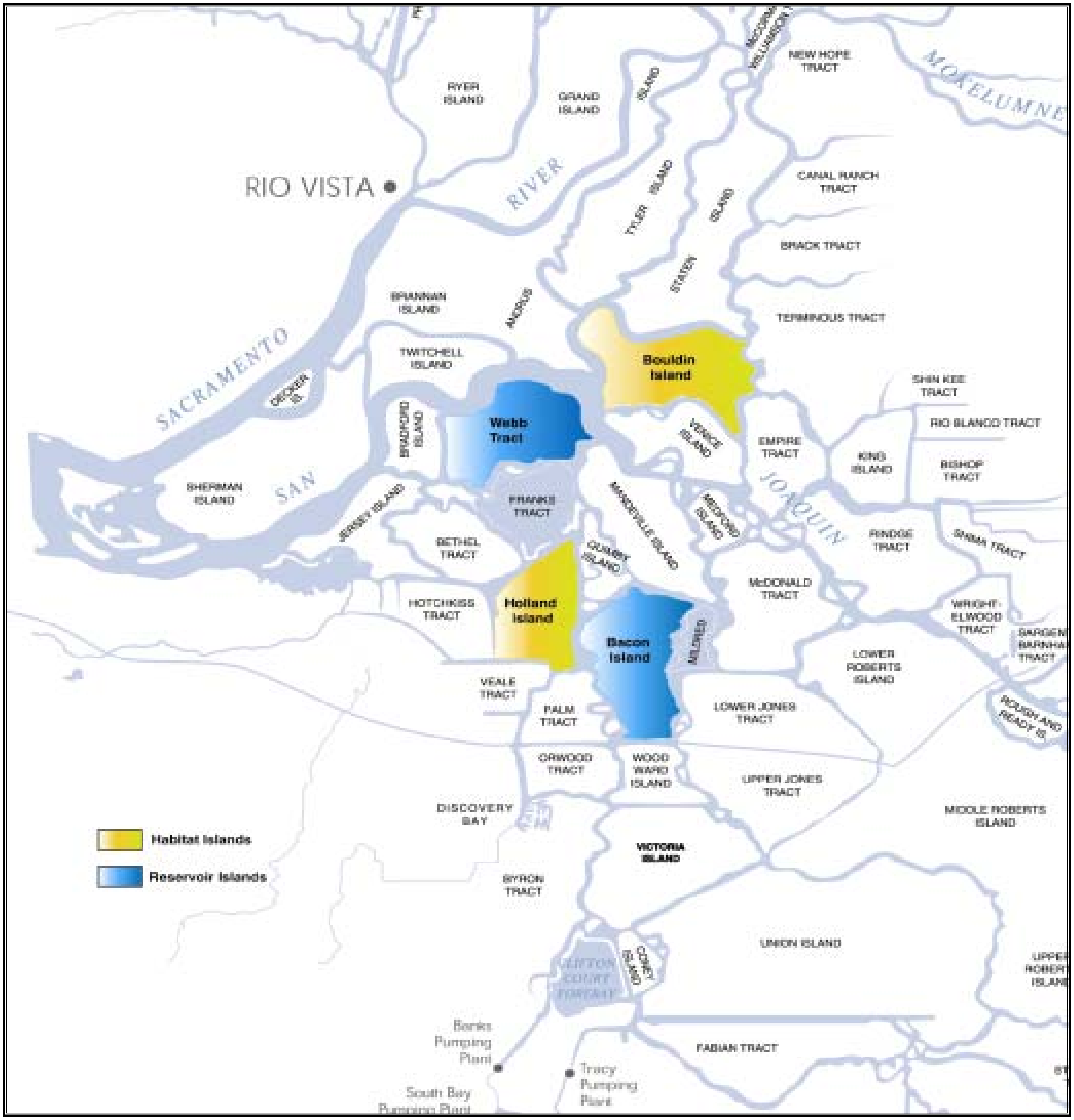
APPENDIX B: FIGURES



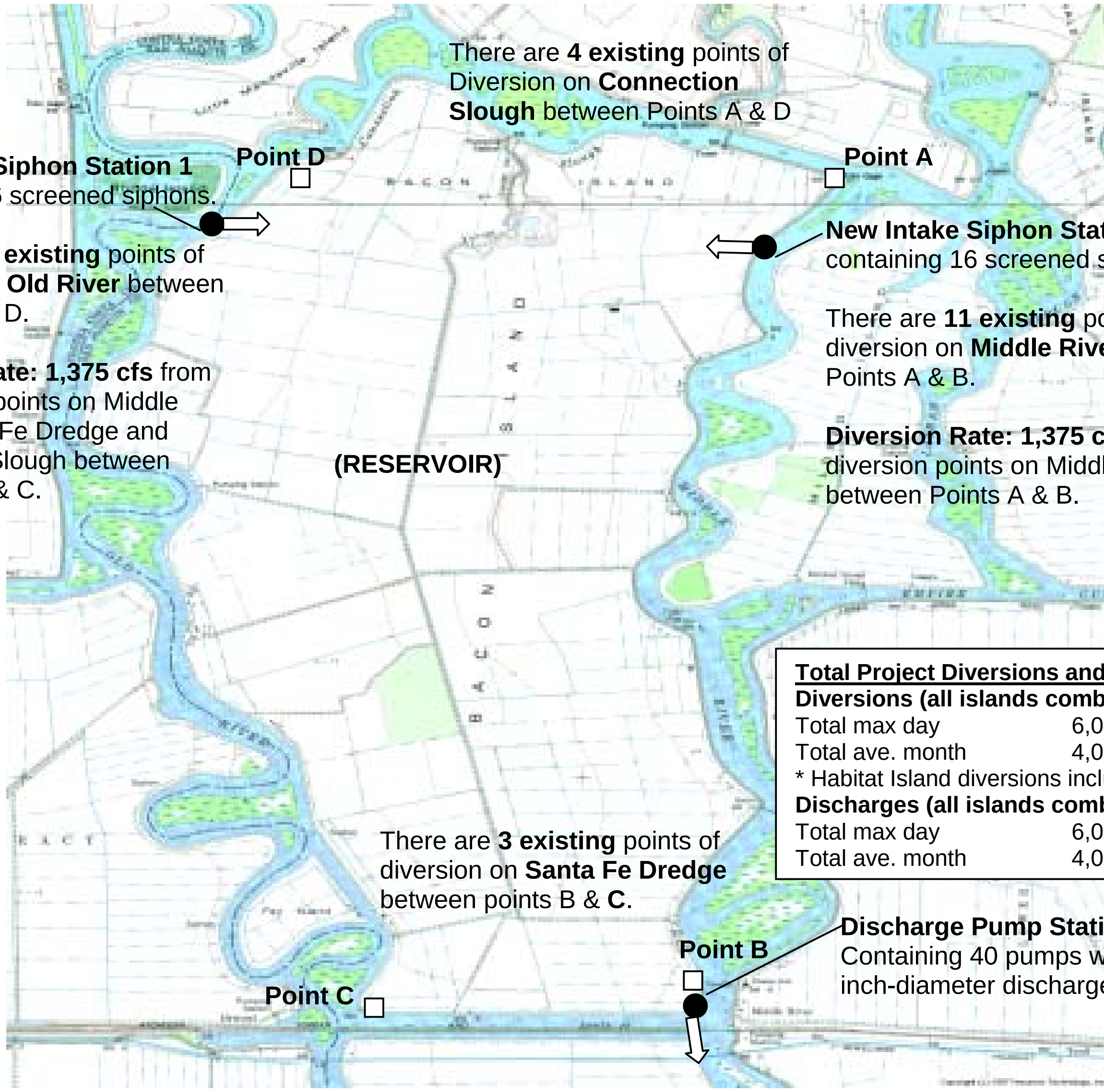
IN-DELTA STORAGE PROGRAM

DELTA WETLANDS PROJECT

PROJECT AREA

Figure 1

Note: A total of 773 dewatering wells are proposed around Webb Tract and Bacon Island to reduce seepage.



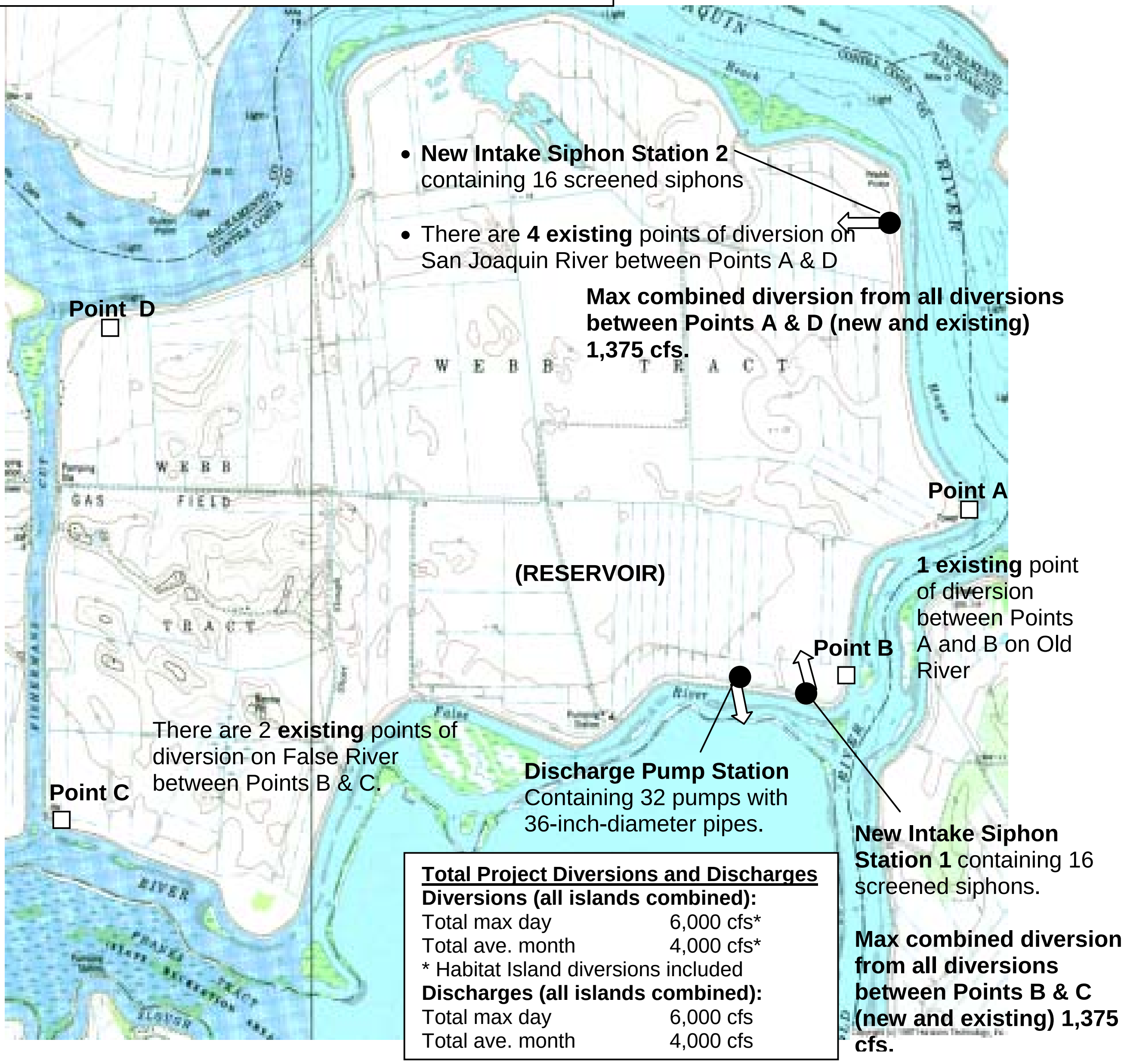
Total Project Diversions and Discharges

Diversions (all islands combined):	
Total max day	6,000 cfs*
Total ave. month	4,000 cfs*
* Habitat Island diversions included	
Discharges (all islands combined):	
Total max day	6,000 cfs
Total ave. month	4,000 cfs

IN-DELTA STORAGE PROGRAM
DELTA WETLANDS PROJECT

Figure 2

Note: A total of 773 dewatering wells are proposed around Webb Tract and Bacon Island to reduce seepage.



<u>Total Project Diversions and Discharges</u>

Diversions (all islands combined):	
Total max day	6,000 cfs*
Total ave. month	4,000 cfs*
* Habitat Island diversions included	
Discharges (all islands combined):	
Total max day	6,000 cfs
Total ave. month	4,000 cfs

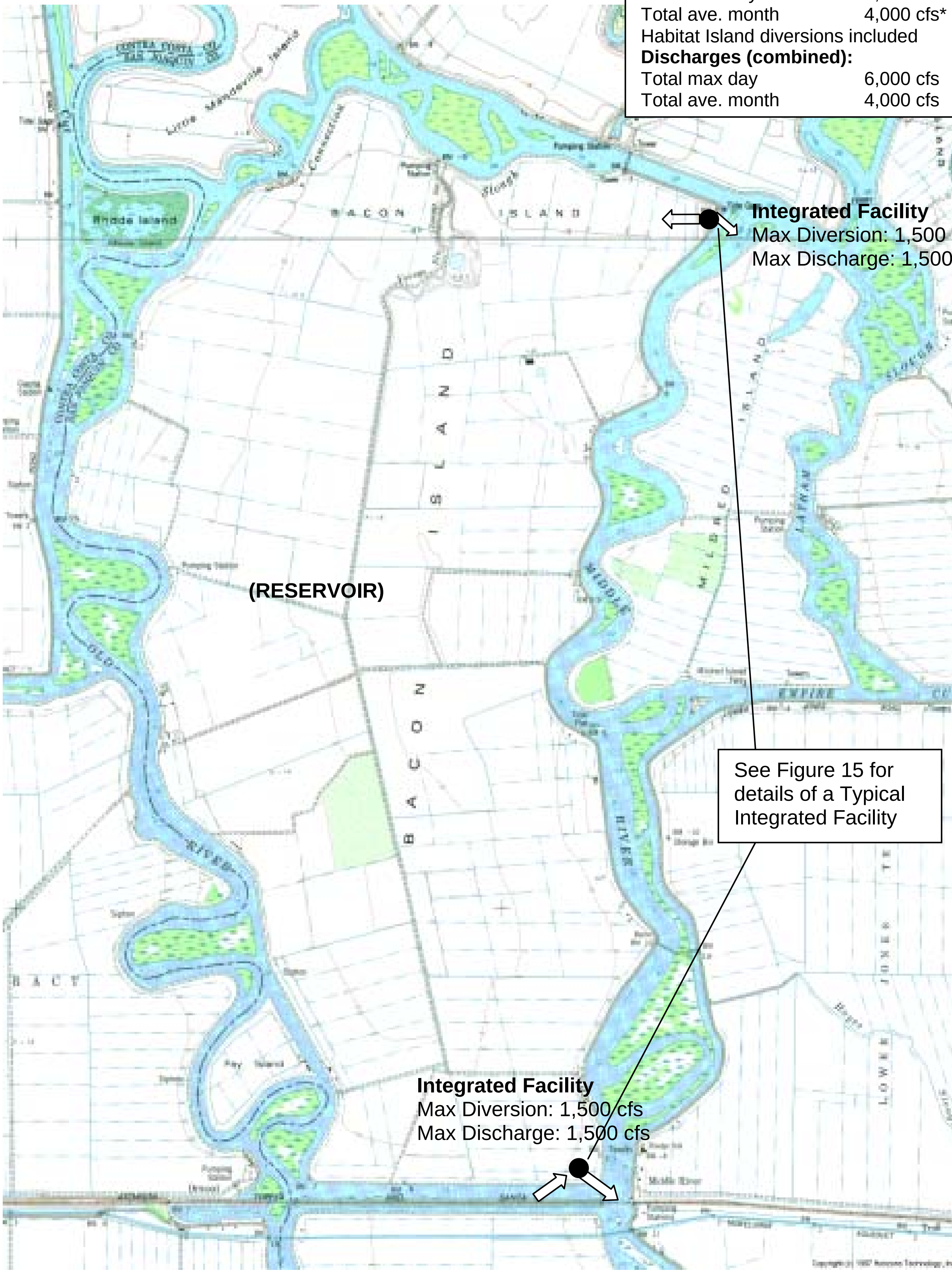
IN-DELTA STORAGE PROGRAM

DELTA WETLANDS PROPOSED PROJECT

Figure 3

Total Project Diversions and Discharges

Diversions (all islands combined):	
Total max day	6,000 cfs*
Total ave. month	4,000 cfs*
Habitat Island diversions included	
Discharges (combined):	
Total max day	6,000 cfs
Total ave. month	4,000 cfs



IN-DELTA STORAGE PROGRAM
RE-ENGINEERED ALTERNATIVE
Figure 4

Total Project Diversions and Discharges

Diversions (all islands combined):

Total max day	6,000 cfs*
Total ave. month	4,000 cfs*

* Habitat Island diversions included

Discharges (all islands combined):

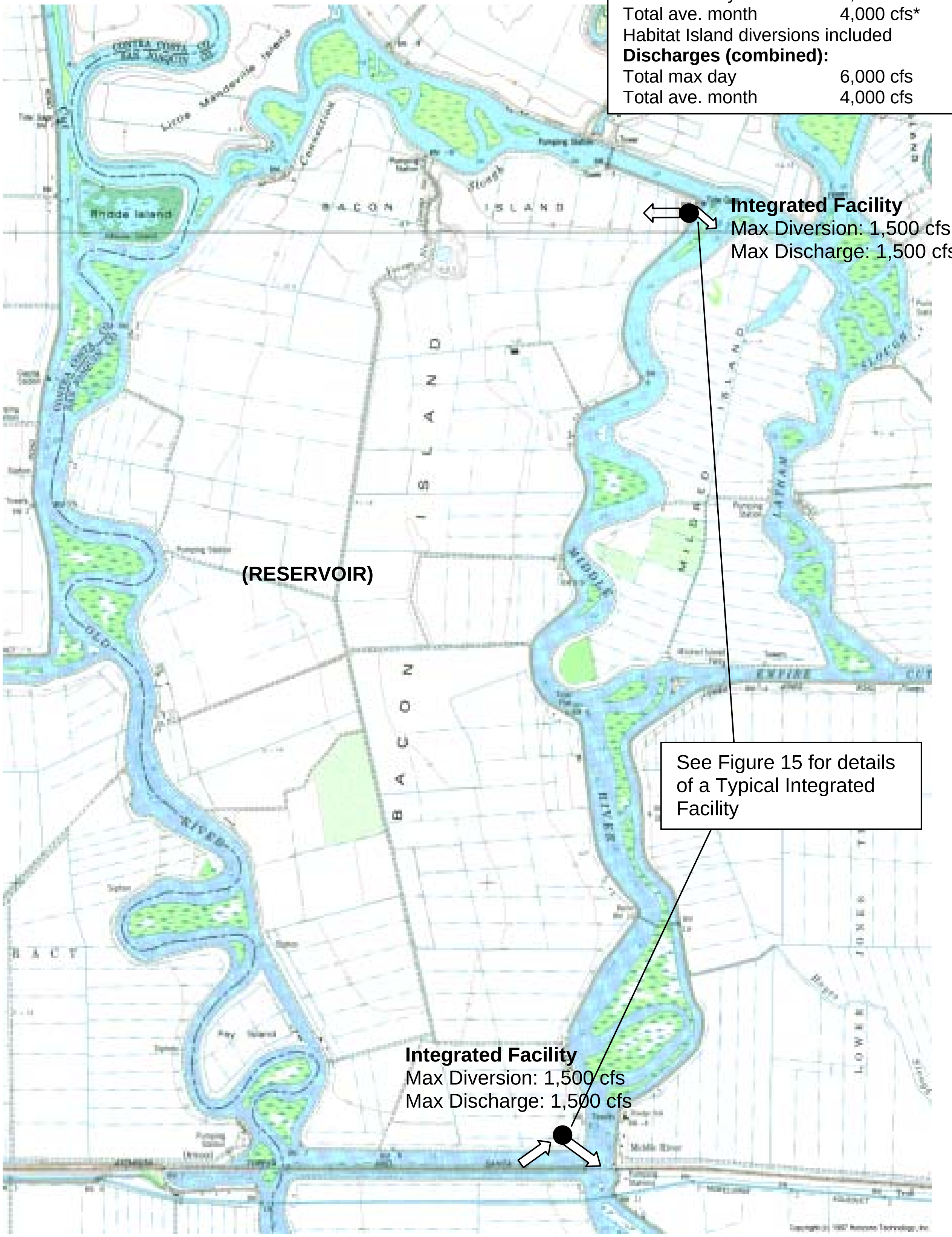
Total max day	6,000 cfs
Total ave. month	4,000 cfs

IN-DELTA STORAGE PROGRAM

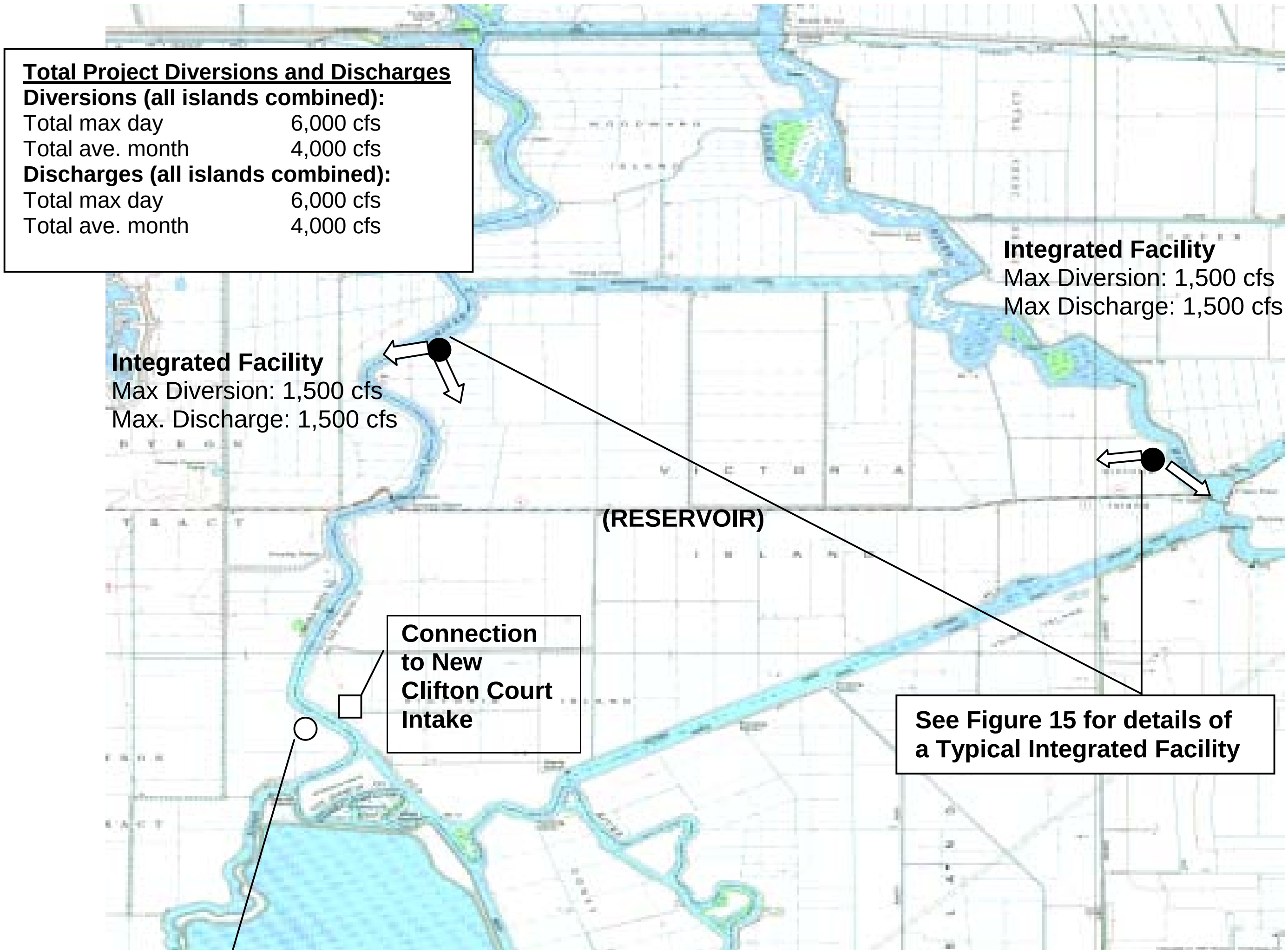
RE-ENGINEERED ALTERNATIVE

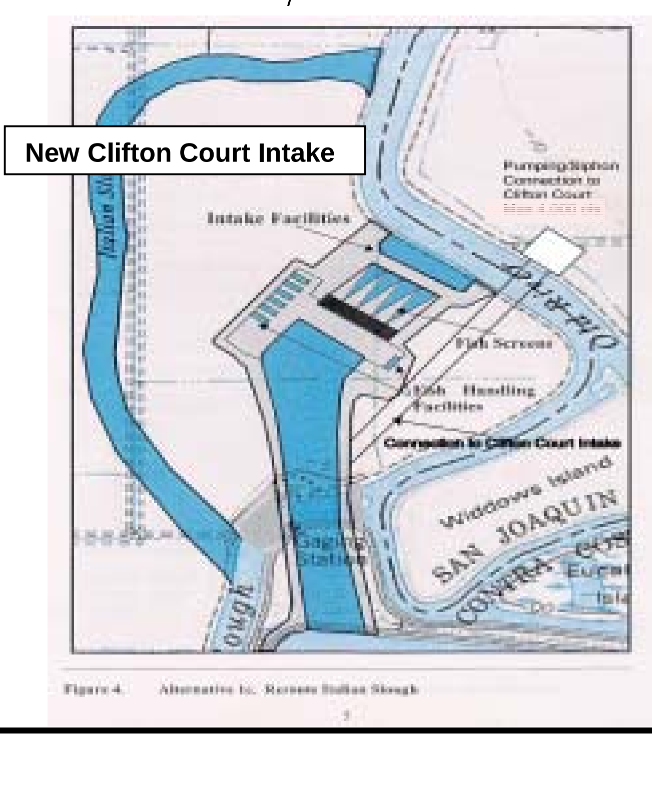
FIGURE 5

Total Project Diversions and Discharges	
Diversions (all islands combined):	
Total max day	6,000 cfs*
Total ave. month	4,000 cfs*
Habitat Island diversions included	
Discharges (combined):	
Total max day	6,000 cfs
Total ave. month	4,000 cfs



IN-DELTA STORAGE PROGRAM
BACON AND VICTORIA OPTION
Figure 6

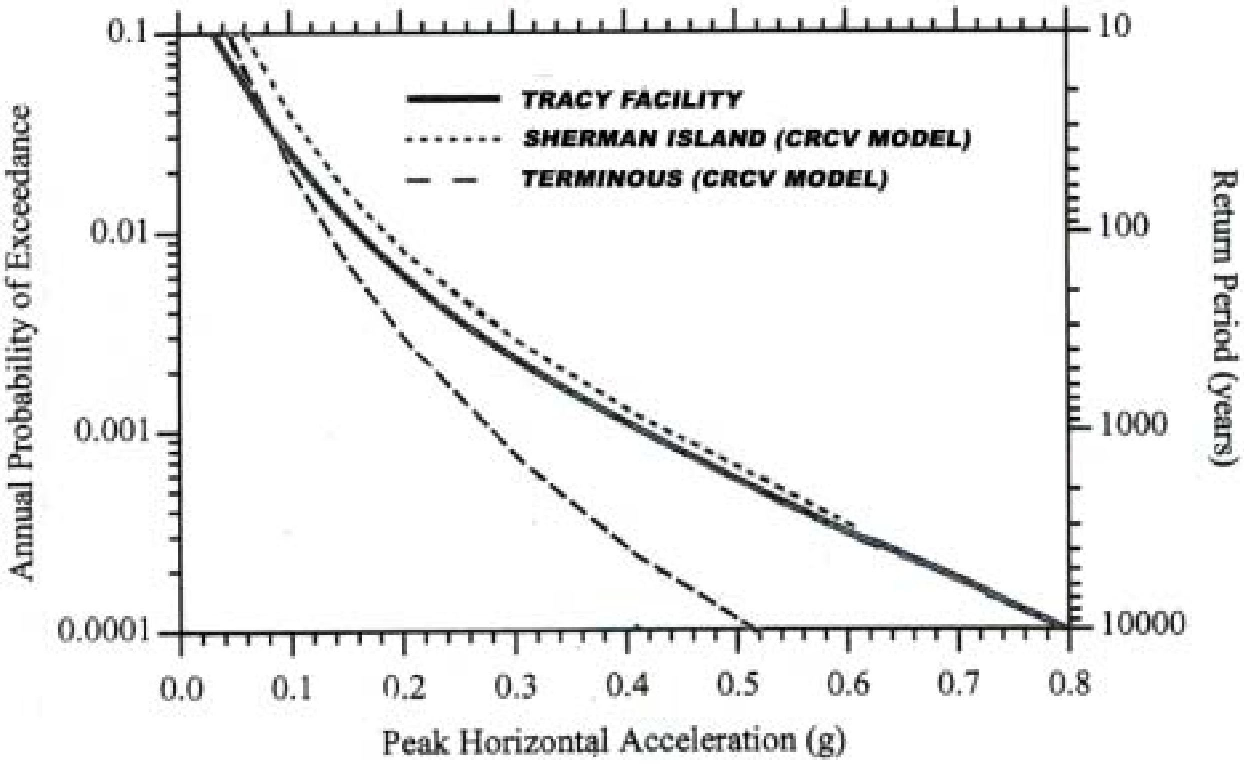




IN-DELTA STORAGE PROGRAM
VICTORIA ISLAND

Figure 7

Figure 8. Peak Horizontal Acceleration Hazard



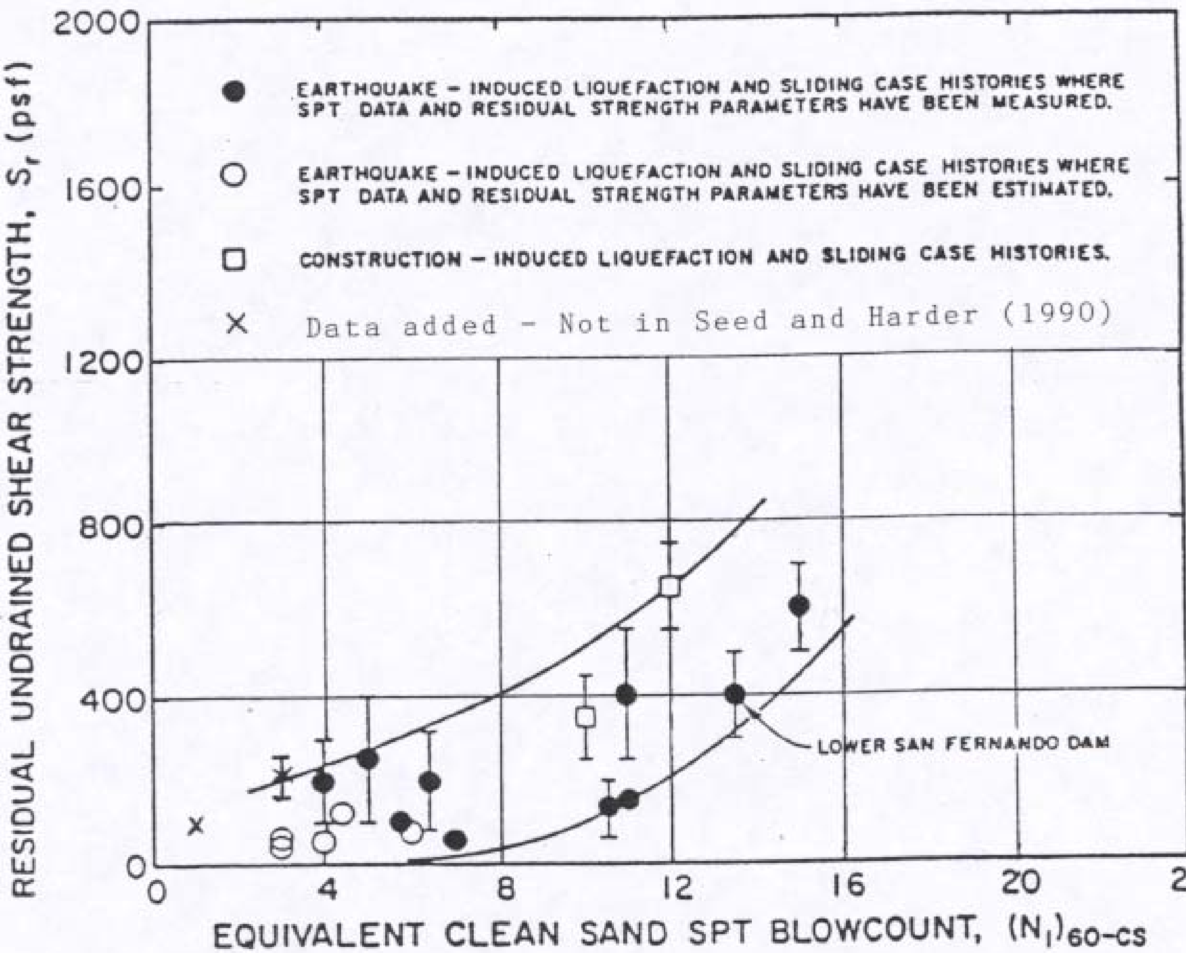


Figure 9
Residual Undrained Shear Strength as a Function of SPT Blowcount

Adjustment to $(N_1)_{60}$ for Fines Content [3, 4]

Fines content	$\Delta(N_1)_{60}$
10%	1
25	2
50	4
75	5

Note: This adjustment is to be used for estimation of S_{ur} only.

Figure 10. Locations of Gauging Stations in the Delta

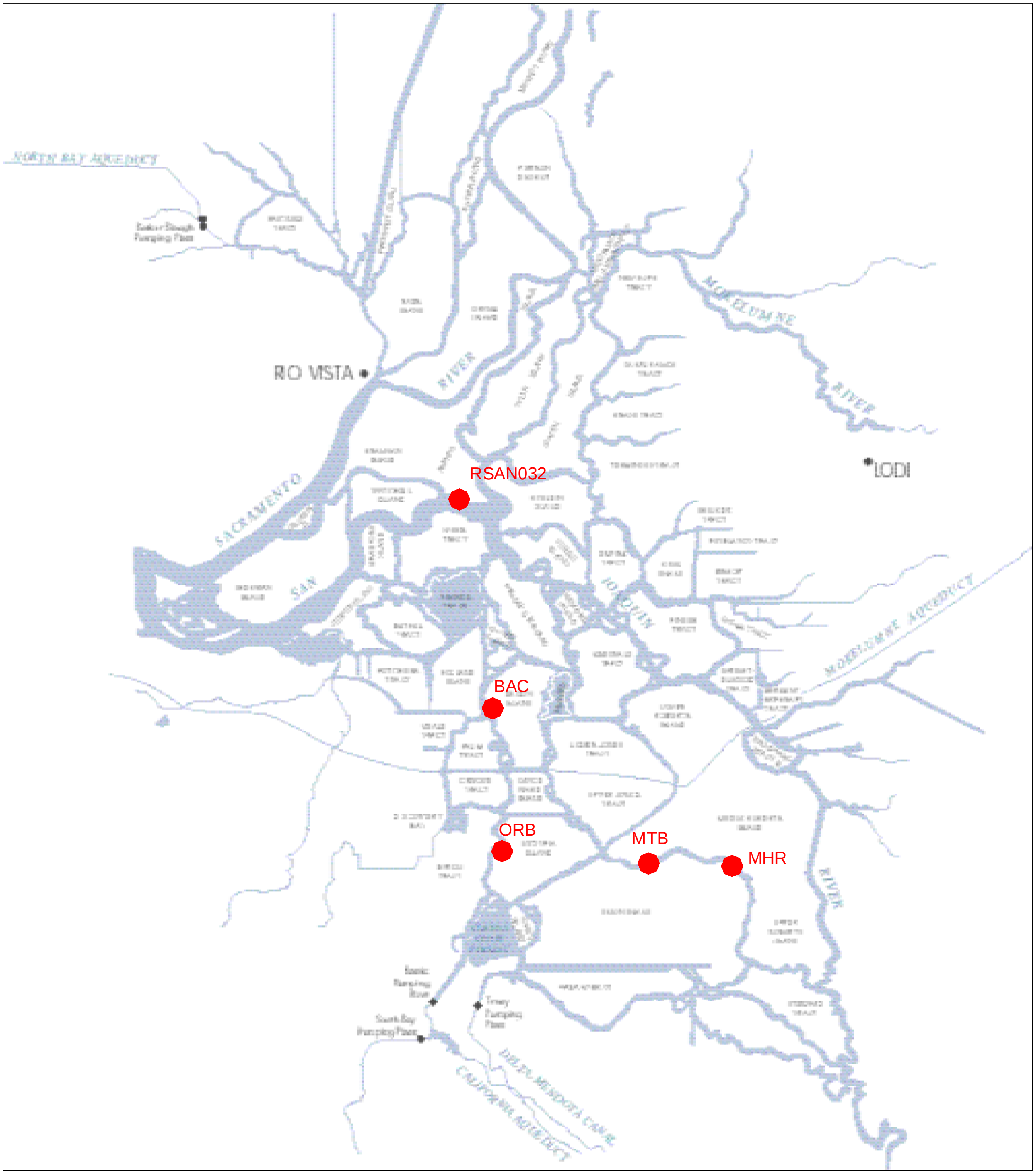
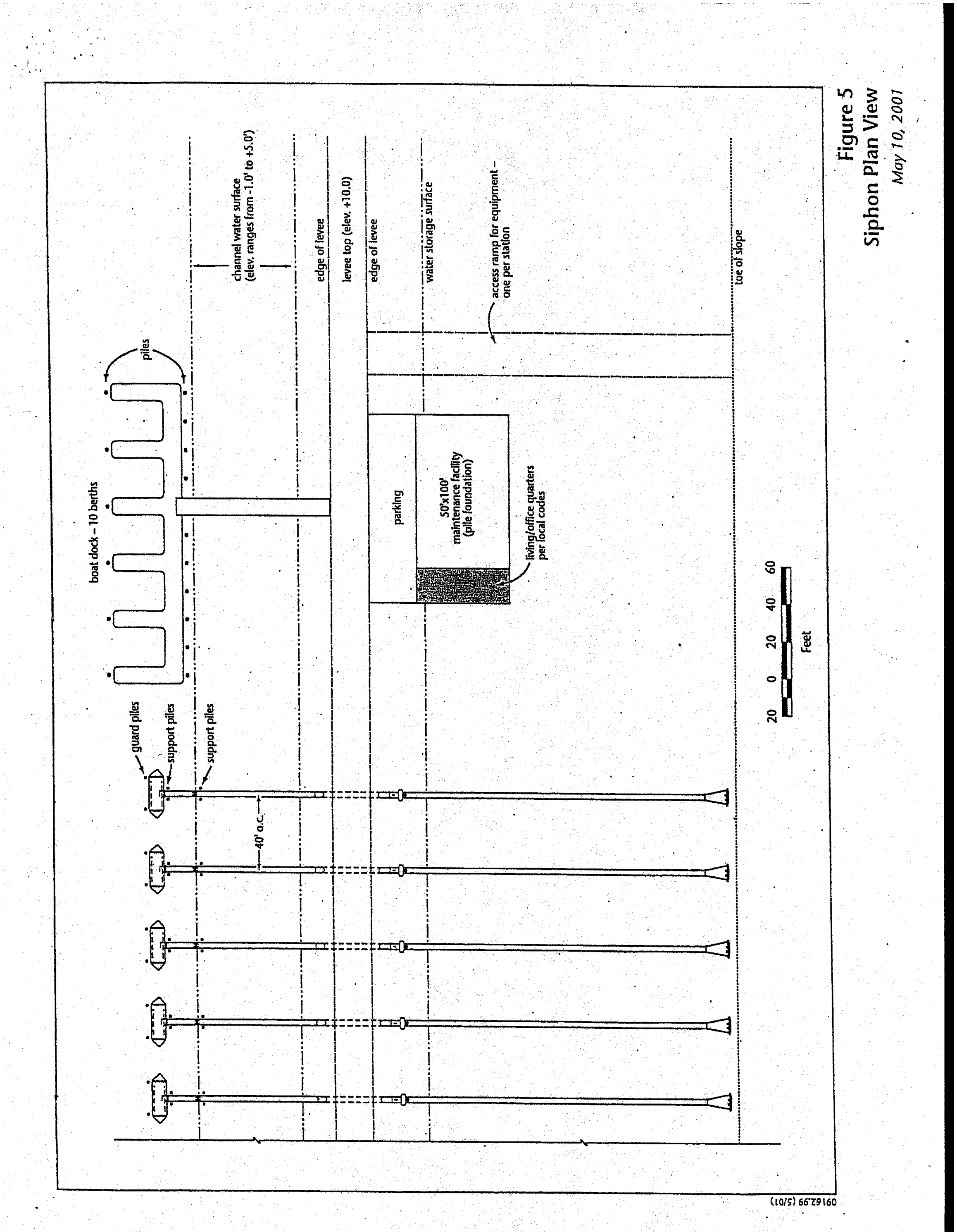


Figure 11. Delta Wetlands Project, Layout Plans and Cross Sections



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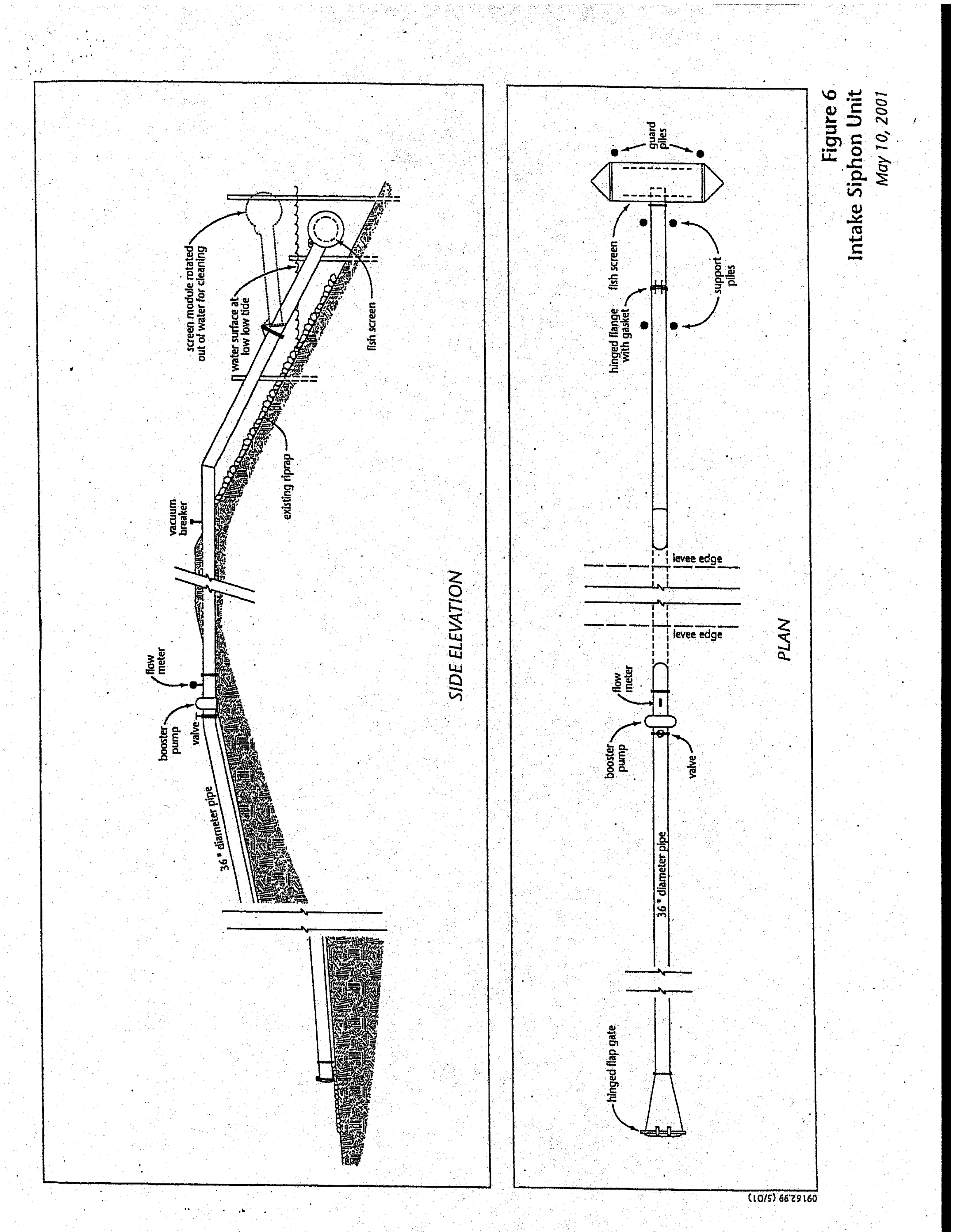

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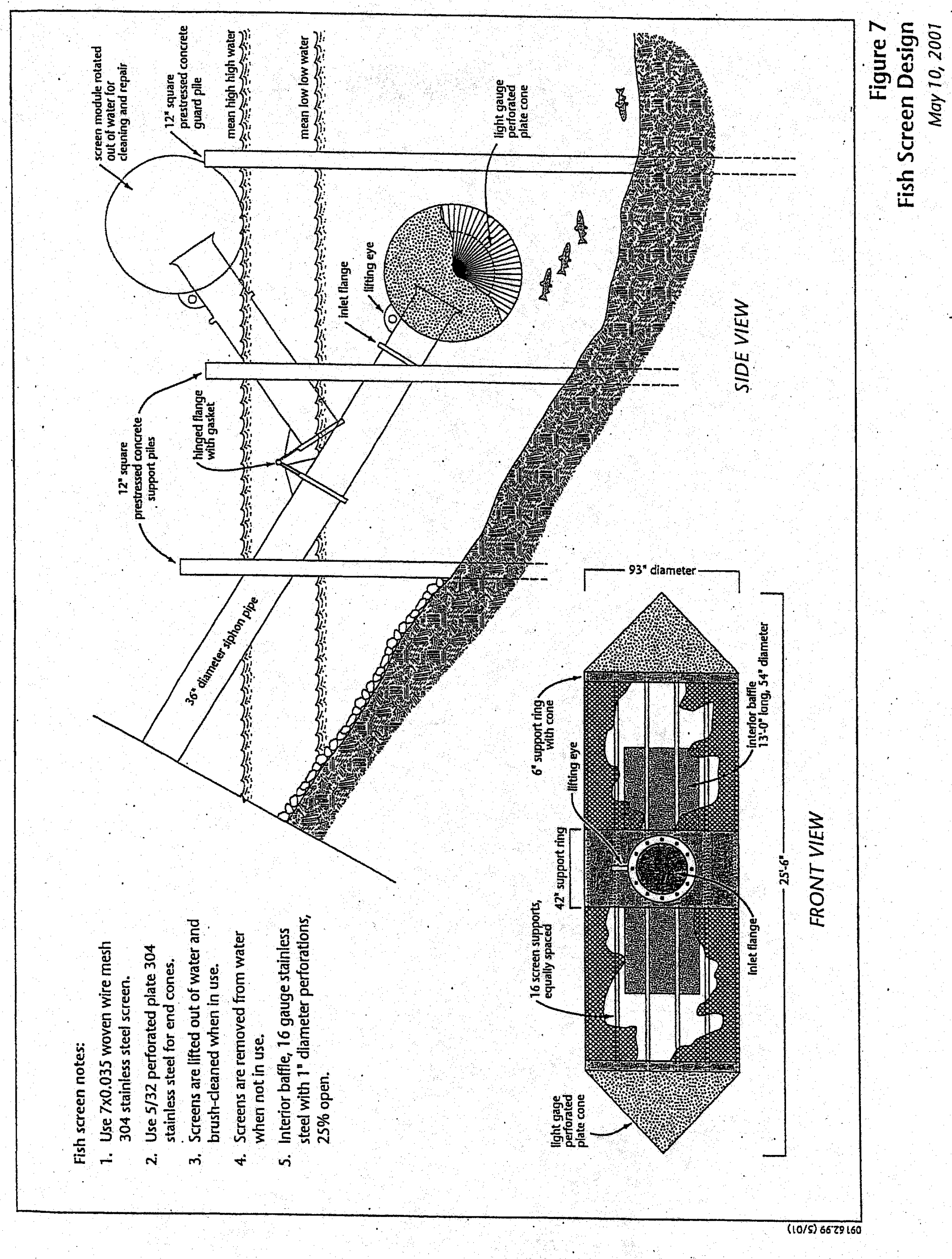

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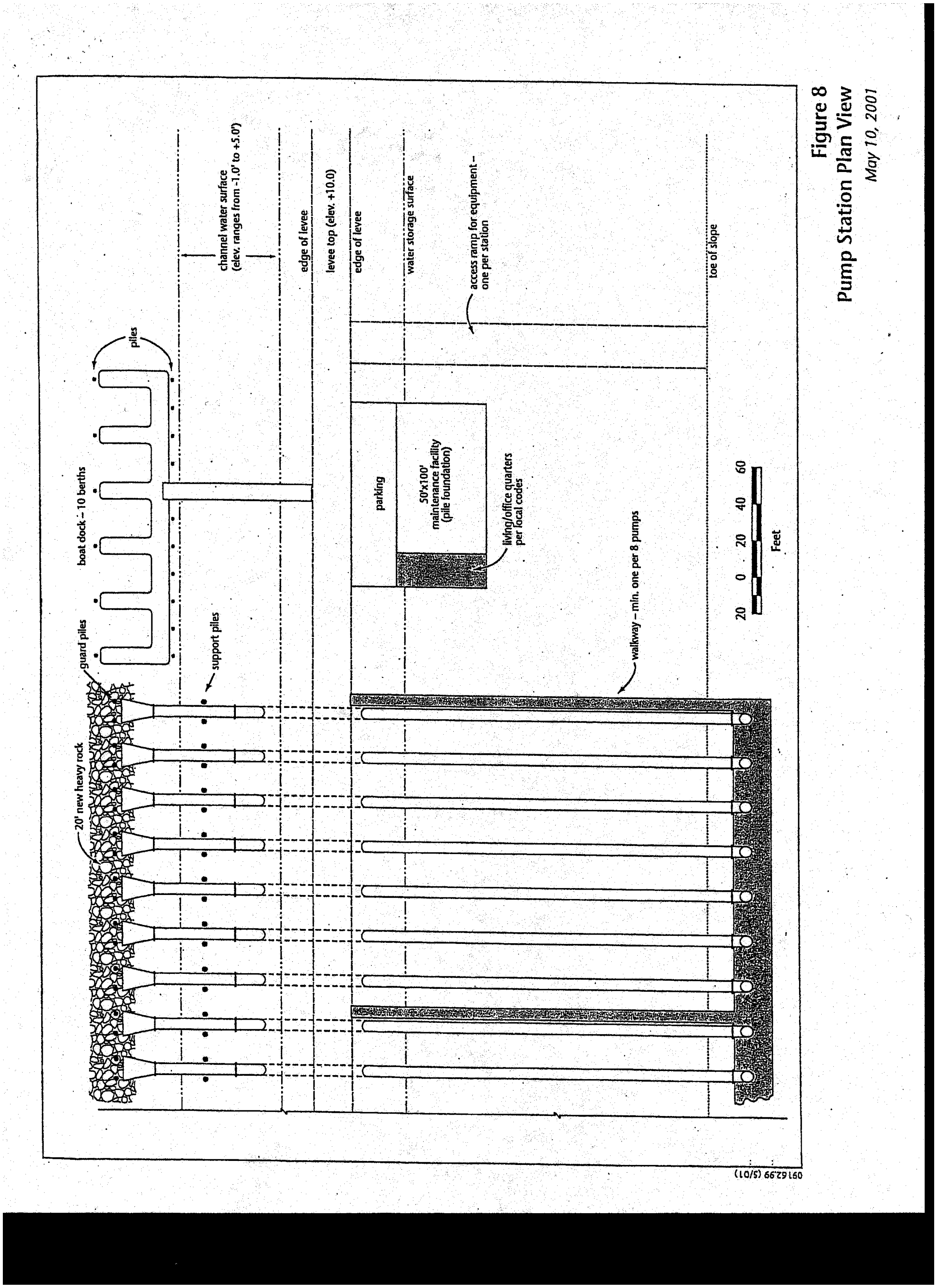

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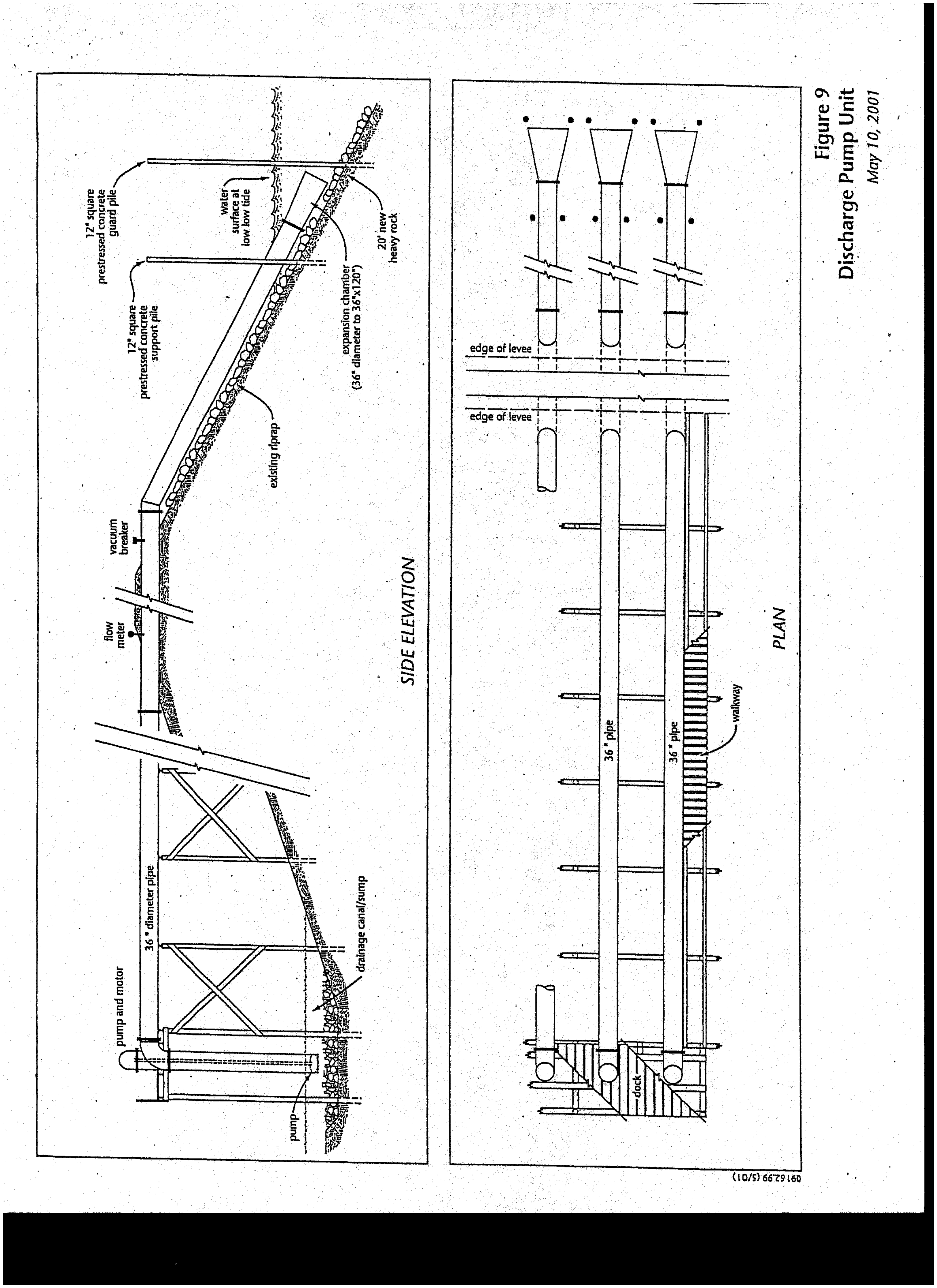


Figure 12. Stationing along Perimeter Embankment of Bacon Island

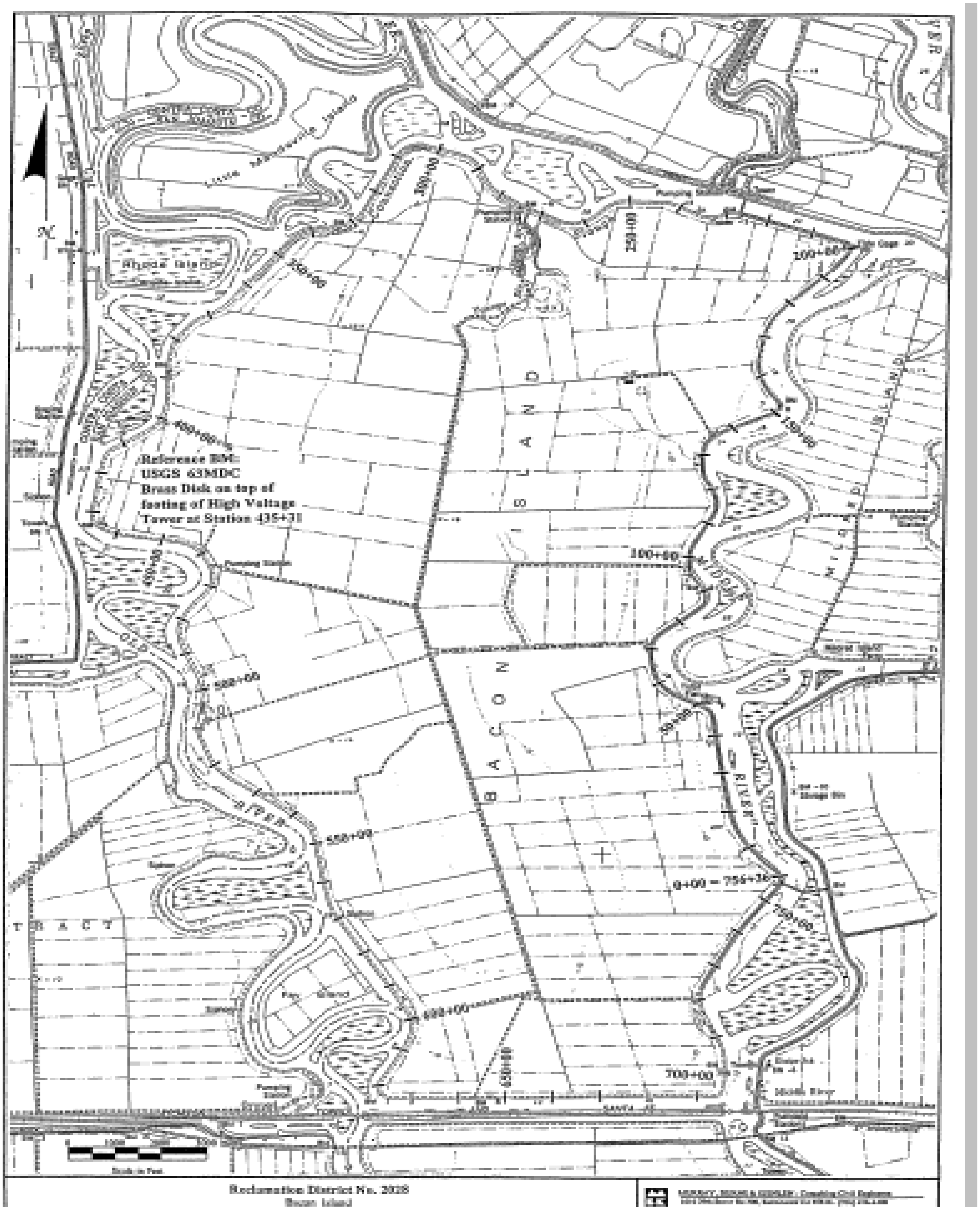


Figure 12. Stationing along Perimeter Embankment of Webb Tract

Figure 12. Stationing along Perimeter Embankment of Victoria Island

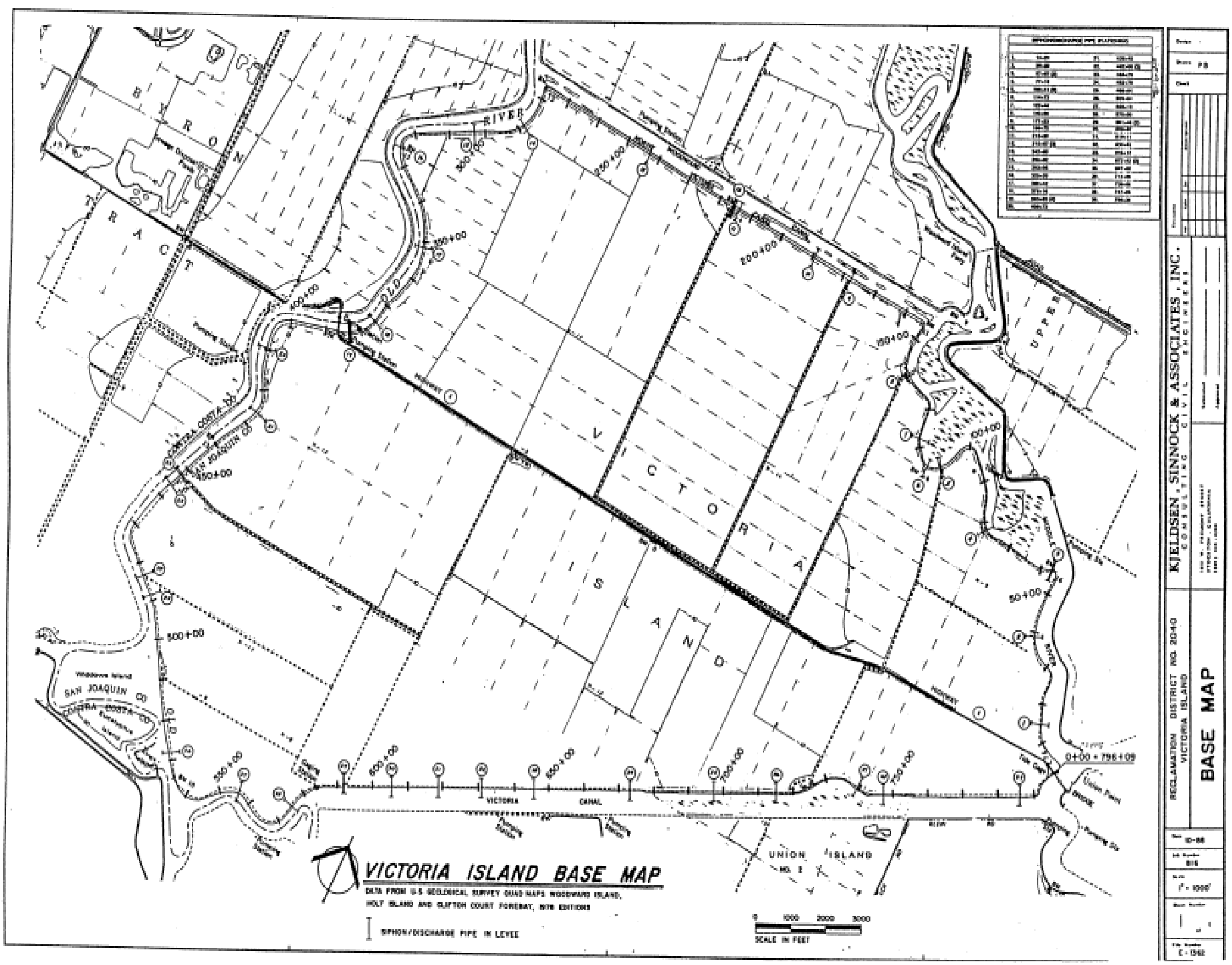
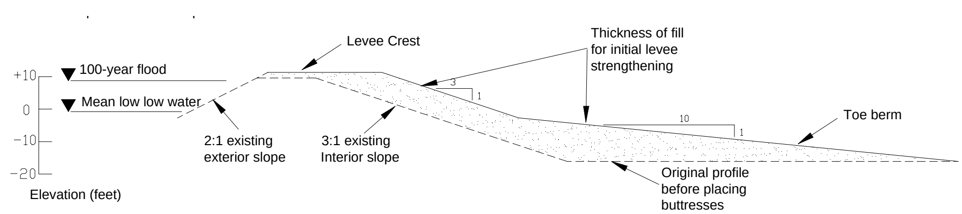
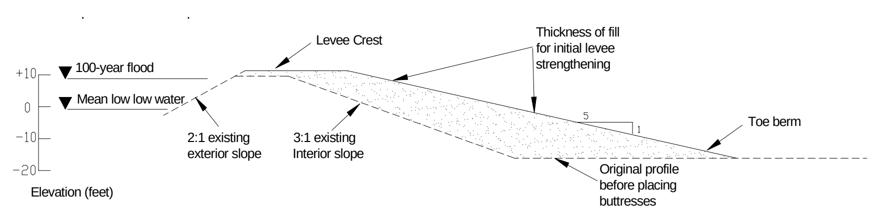


Figure 13: Delta Wetlands Proposed Embankment Cross Sections



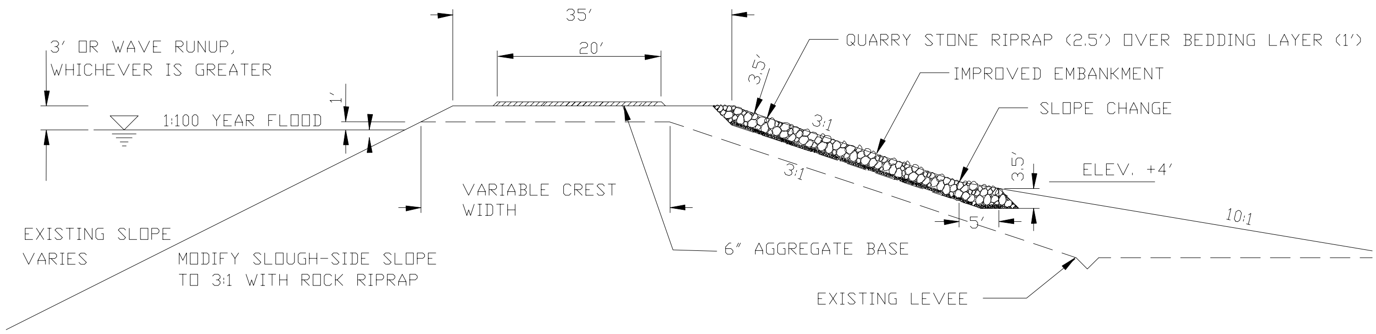
a) Broken-slope Section



b) Constant-slope Section

Note: Not to Scale

Figure 14: Re-Engineered Embankment Cross-Section



TYPICAL INTEGRATED FACILITY

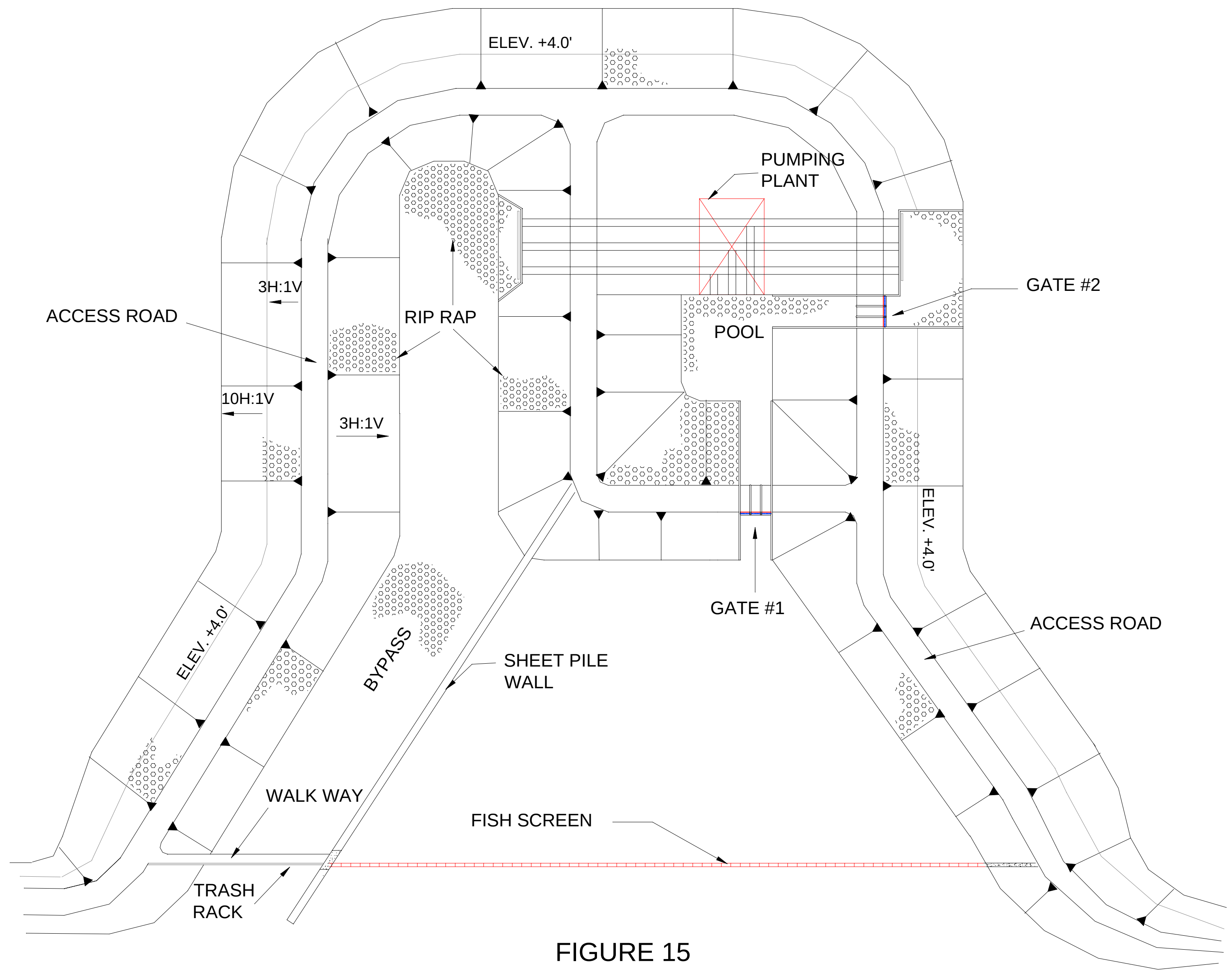


FIGURE 15

SCALE: 1" = 200'

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					DRAWN	REVIEWED	APPROVAL RECOMMENDED:
					SUBMITTED:		APPROVED: DATE:

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THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
DIVISION OF PLANNING AND LOCAL ASSISTANCE
INTEGRATED STORAGE INVESTIGATIONS

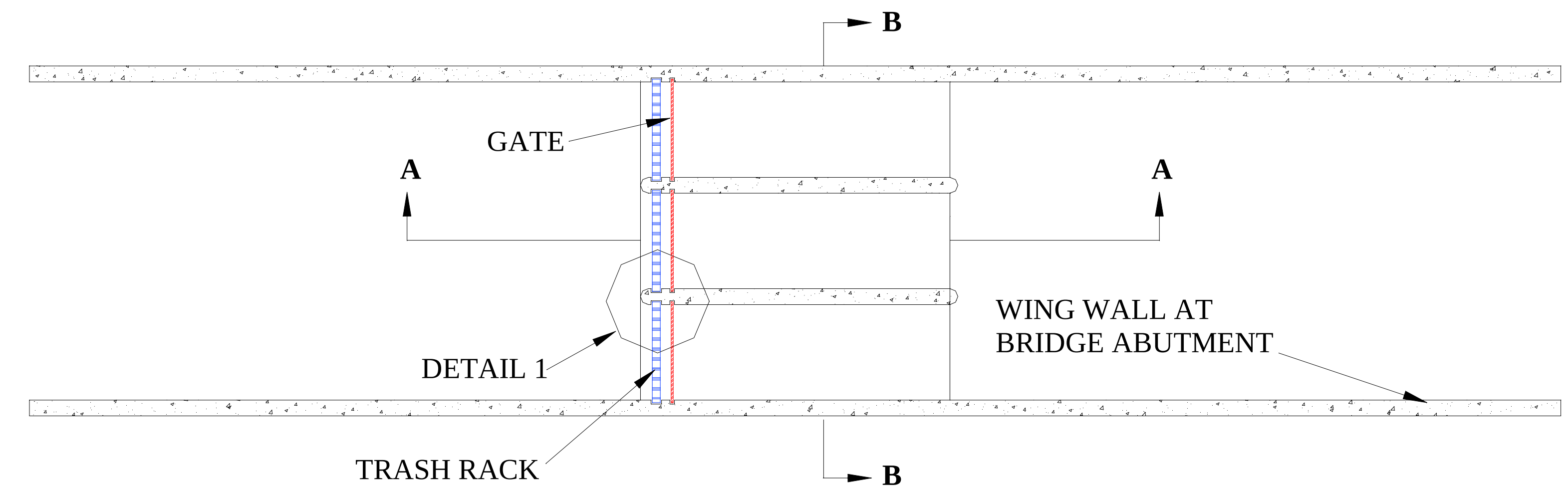
IN DELTA STORAGE PROGRAM

SPEC. No.
DRAWING No.
REV. SHEET No.

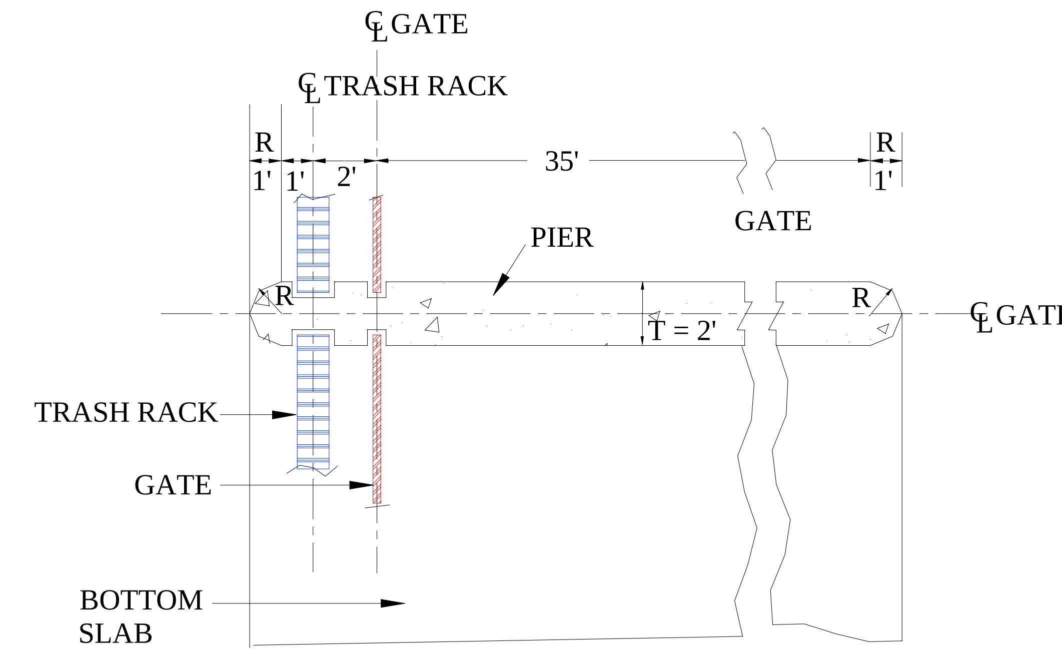
DESIGNFILESPECIFICATION

DATE
DESIGNFILESPECIFICATION

FIGURE 16. GATE STRUCTURE

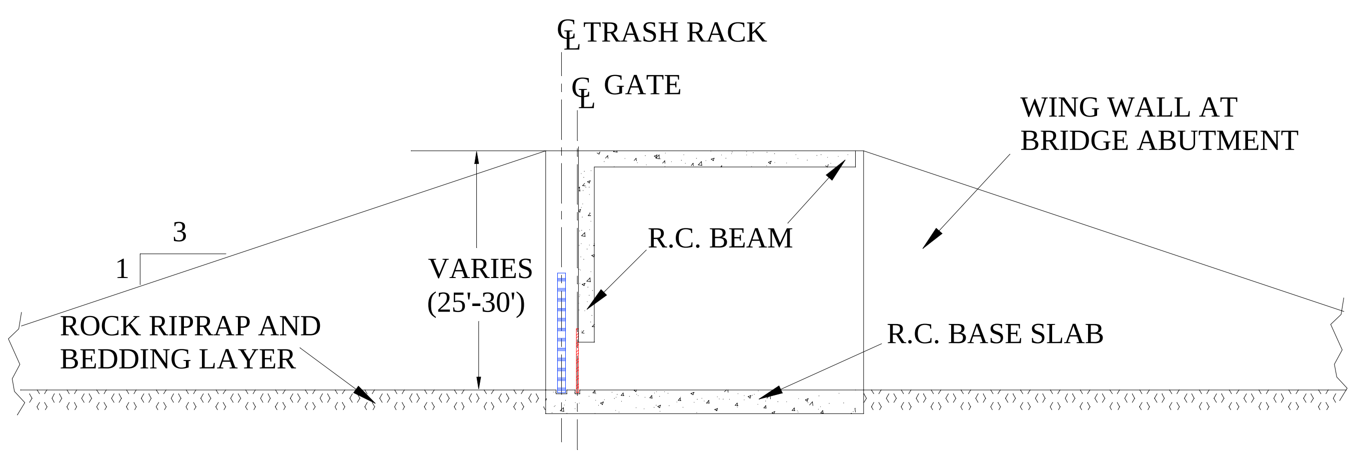


GATE STRUCTURE -PLAN VIEW

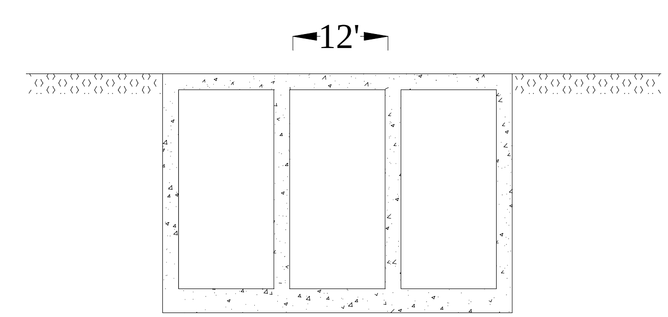


DETAIL 1 (TYP.)

(Scale = 4X)



SECTION AA



SECTION BB

SCALE - 1"=25'

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DRAWN | REVIEWED | APPROVAL RECOMMENDED:
SUBMITTED: | APPROVED: | DATE:

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DRAWING No.
REV. | SHEET No.

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DATE
DESIGNFILESPECIFICATION

WEBB TRACT
(SAN JOAQUIN RIVER INTEGRATED FACILITY)

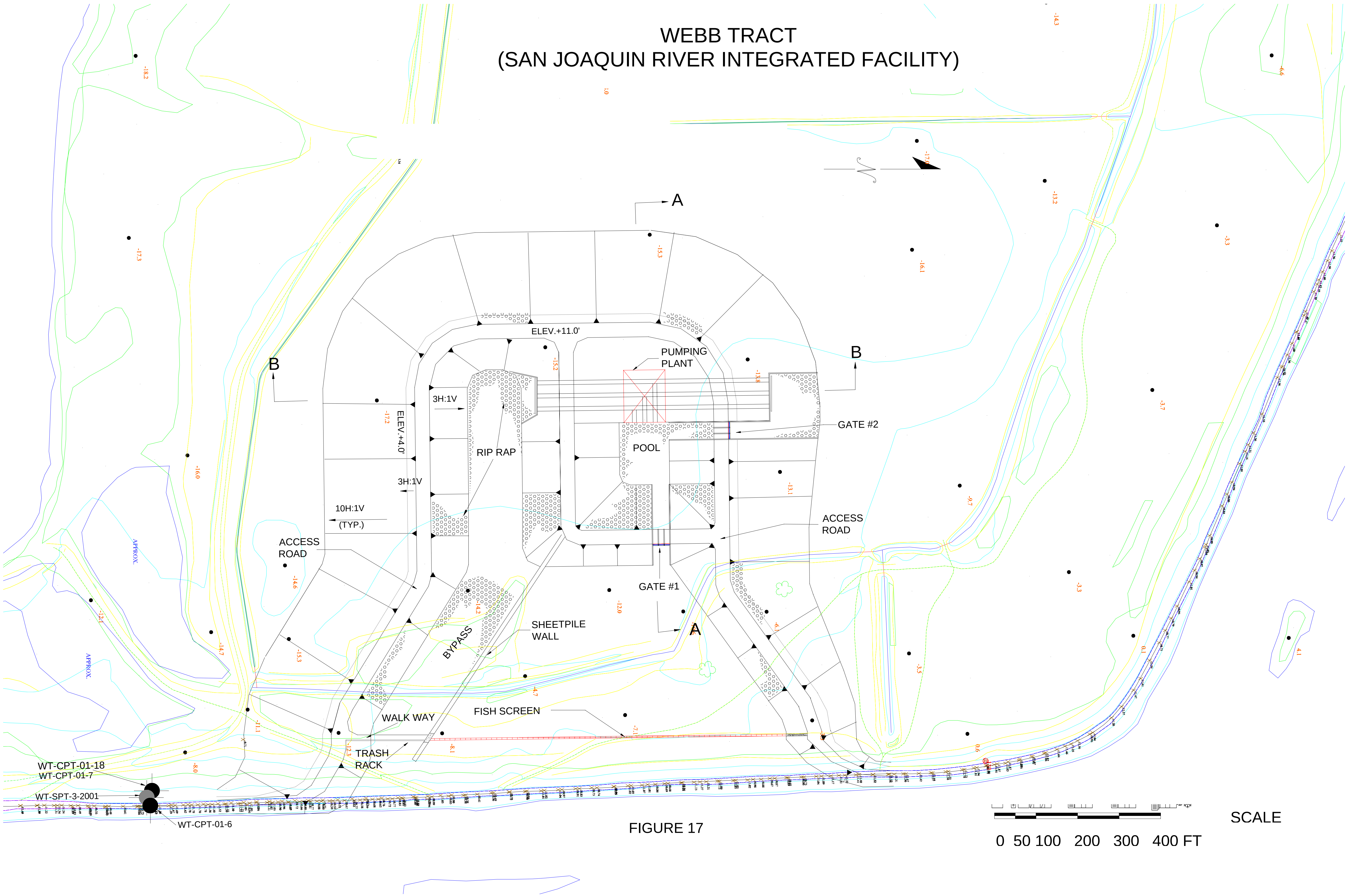
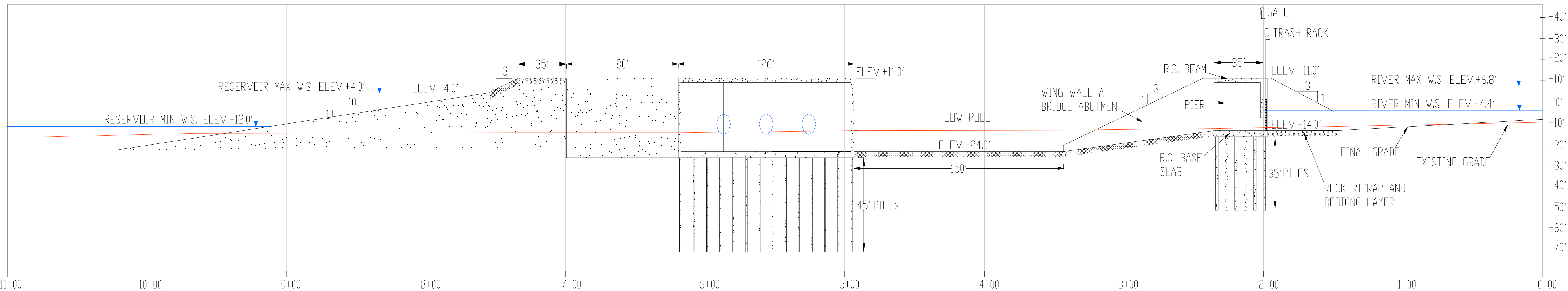


FIGURE 17

WEBB TRACT
(San Joaquin River Integrated Facility)

SECTION -AA



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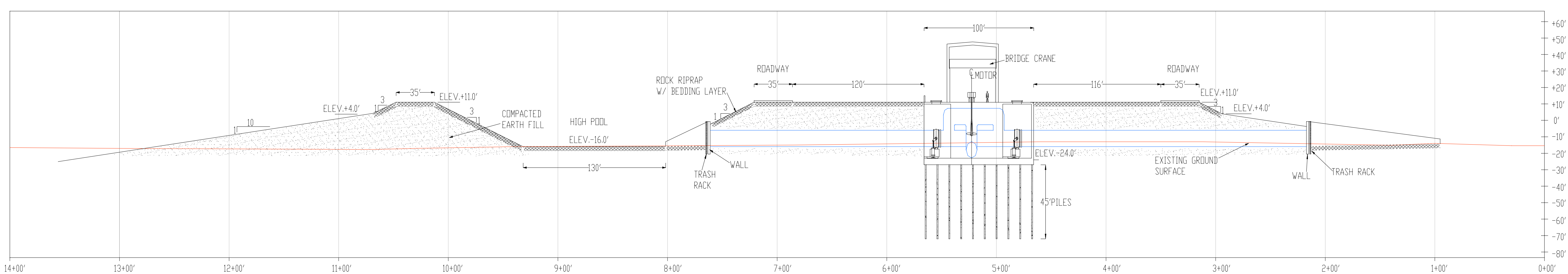

FIGURE 18

REV.	DATE	DESCRIPTION	SUB.	APP'D	DESIGNED / CHECKED / DRAWN / REVIEWED / SUBMITTED	APPROVAL RECOMMENDED: / APPROVAL RECOMMENDED: / APPROVED: DATE:

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THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
DIVISION OF PLANNING AND LOCAL ASSISTANCE
INTEGRATED STORAGE INVESTIGATIONS

IN DELTA STORAGE PROGRAM

SPEC. No.
DRAWING No.
REV. SHEET No.

WEBB TRACT
(FLASE RIVER INTEGRATED FACILITY)

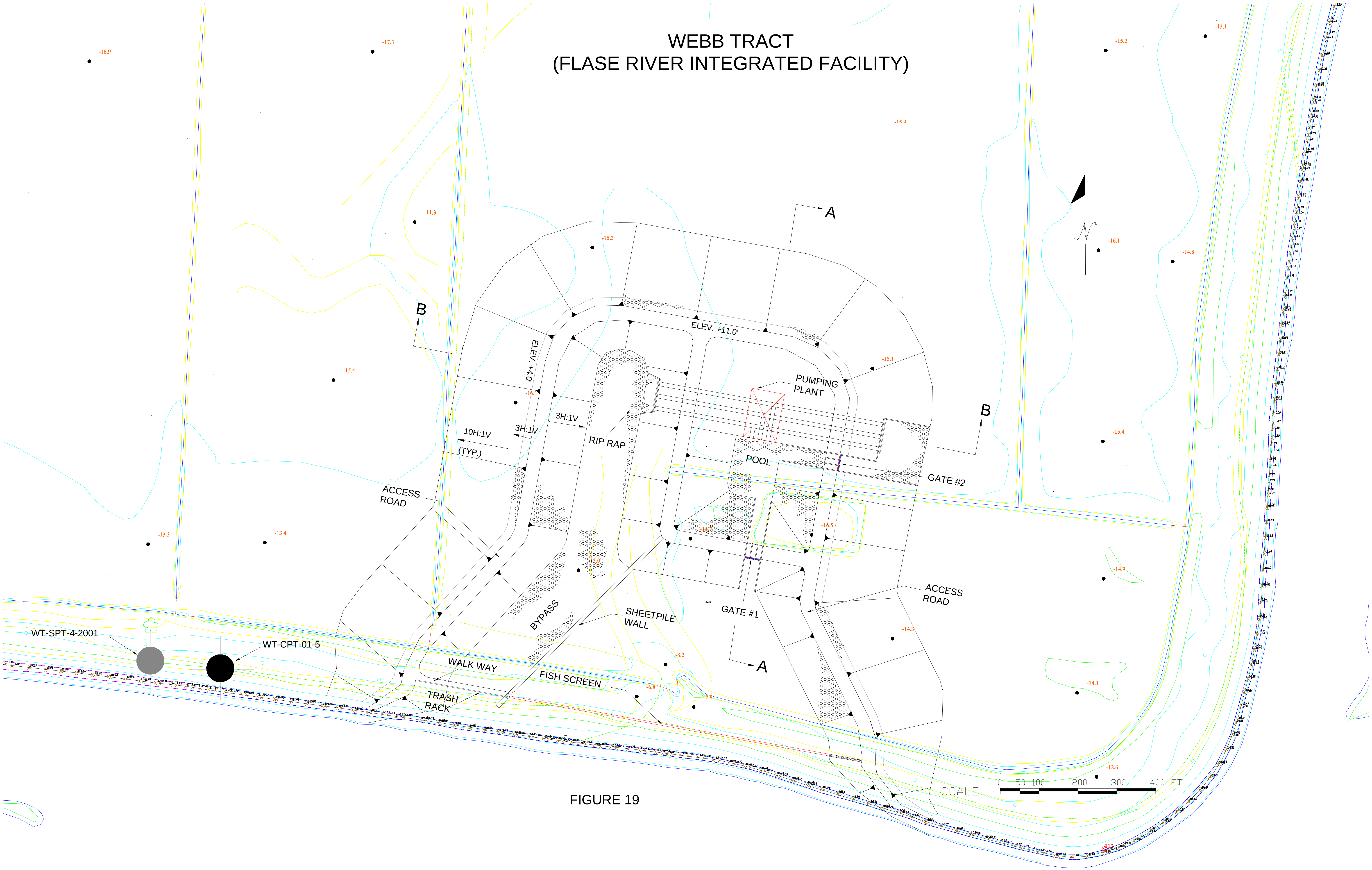
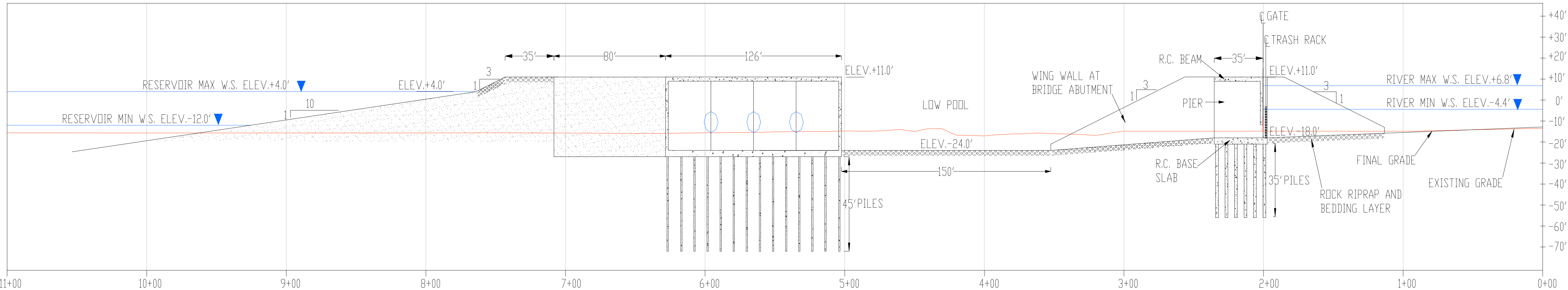


FIGURE 19

WEBB TRACT
(False River Integrated Facility)

SECTION -AA



SECTION -BB

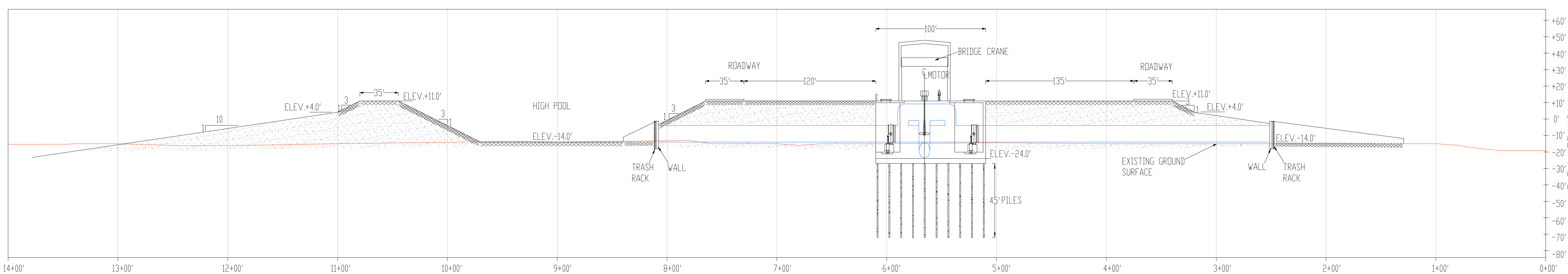


FIGURE 20

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BACON ISLAND
(MIDDLE RIVER INTEGRATED FACILITY)

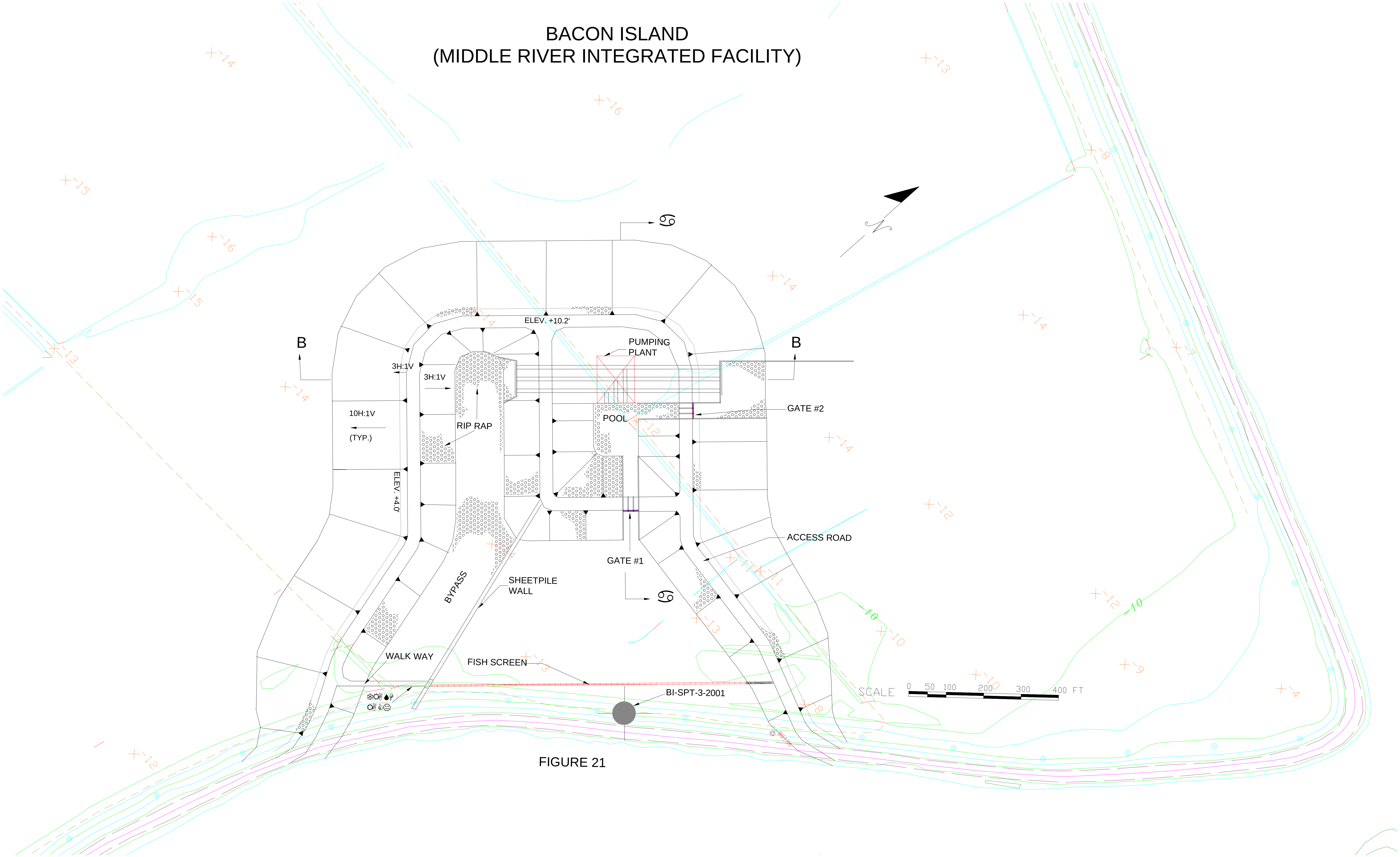
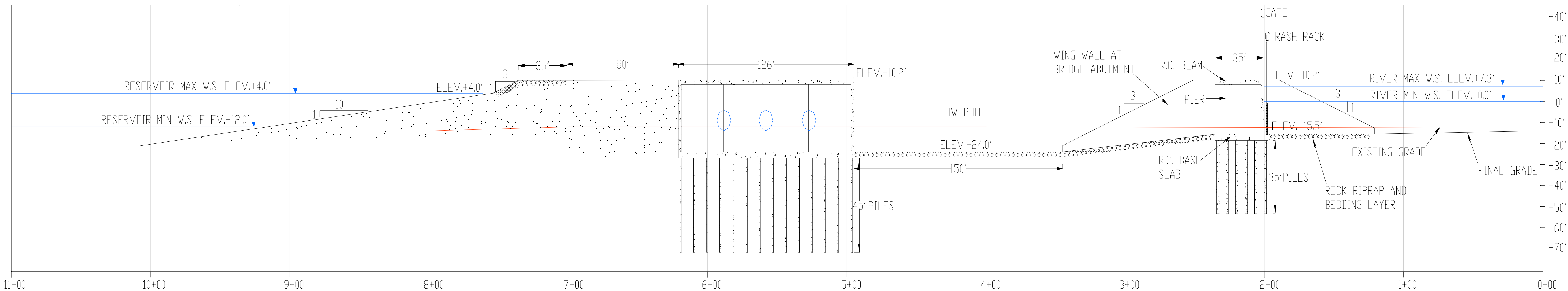


FIGURE 21

BACON ISLAND
(Middle River Integrated Facility)

SECTION -AA



SECTION -BB

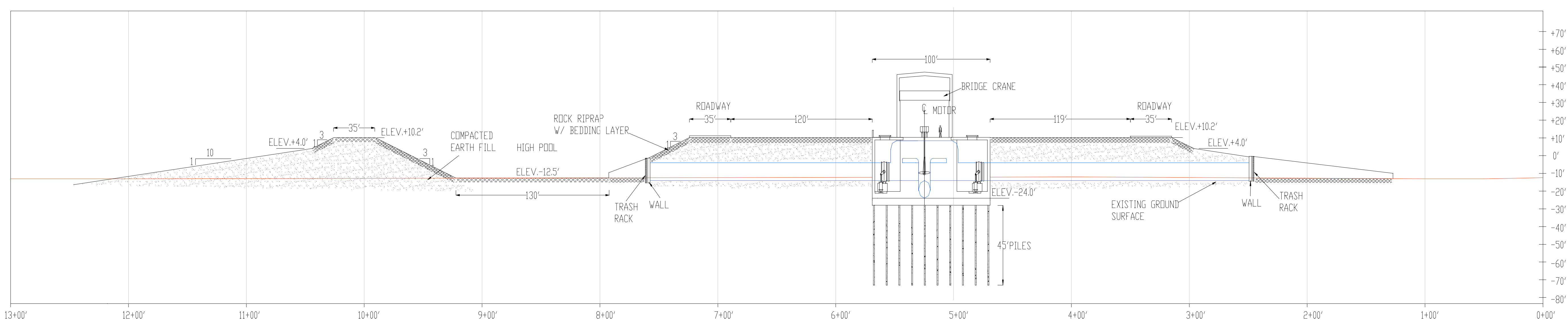


FIGURE 22

REV. DATE DESCRIPTION SUB. APP'D

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REV. SHEET No.

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DESIGNFILESPECIFICATION

BACON ISLAND
(SANTA FE CUT INTEGRATED FACILITY)

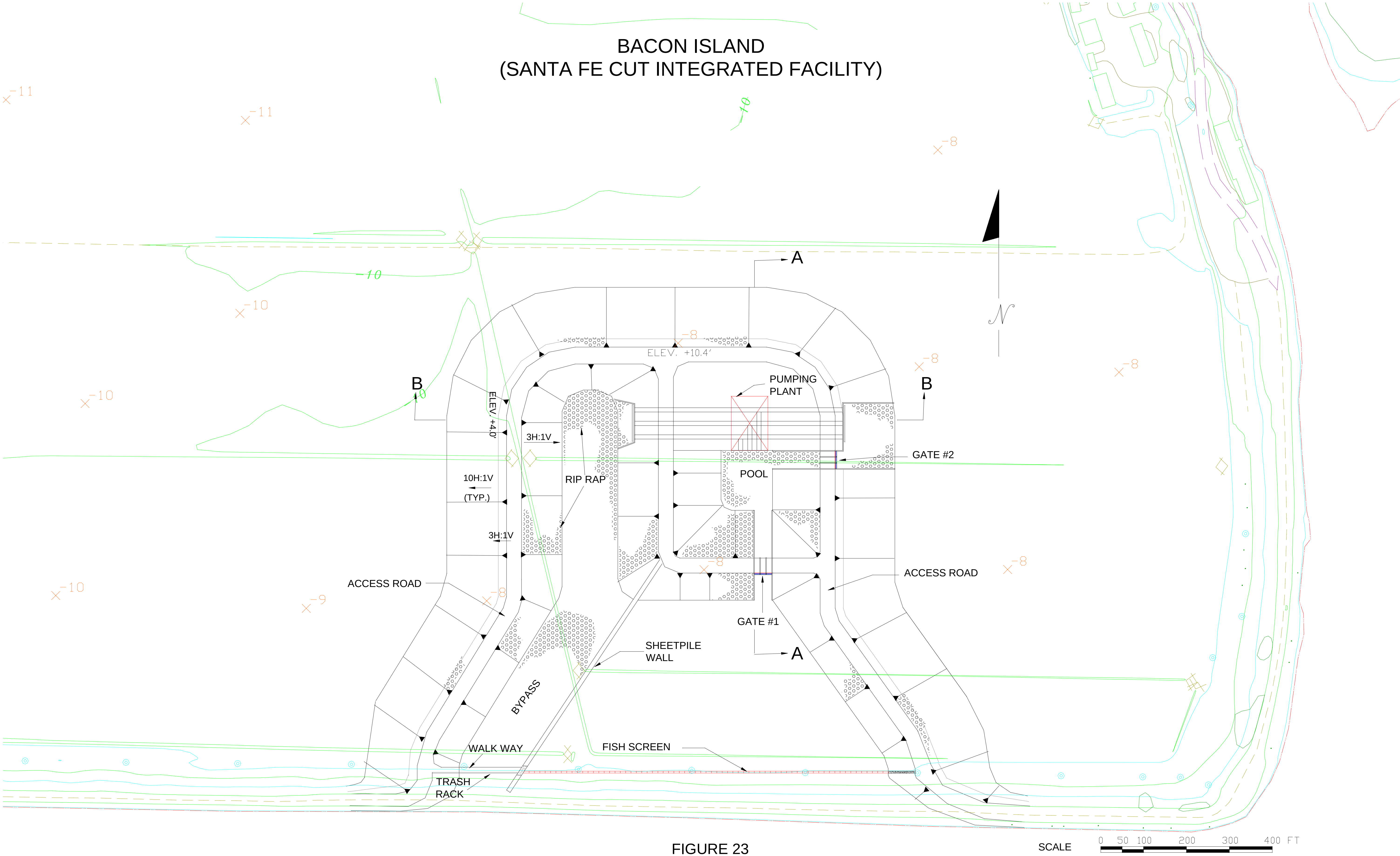
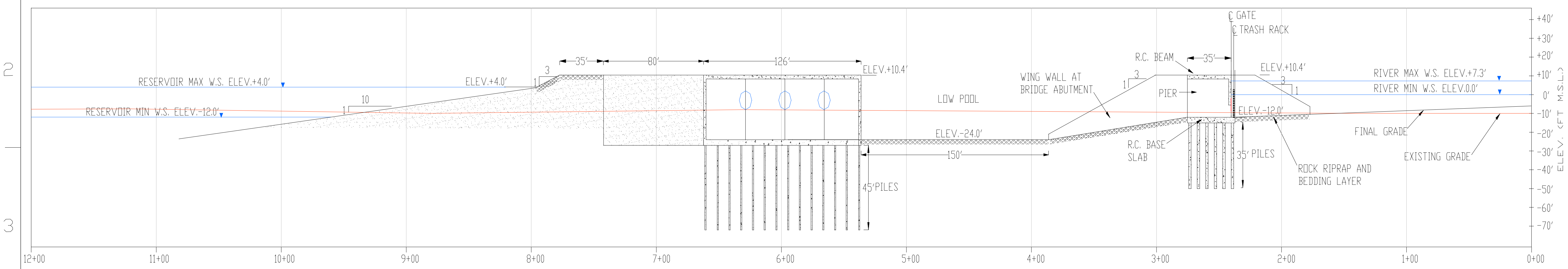


FIGURE 23

BACON ISLAND
(Santa Fe Cut Integrated Facility)

SECTION -AA



SECTION -BB

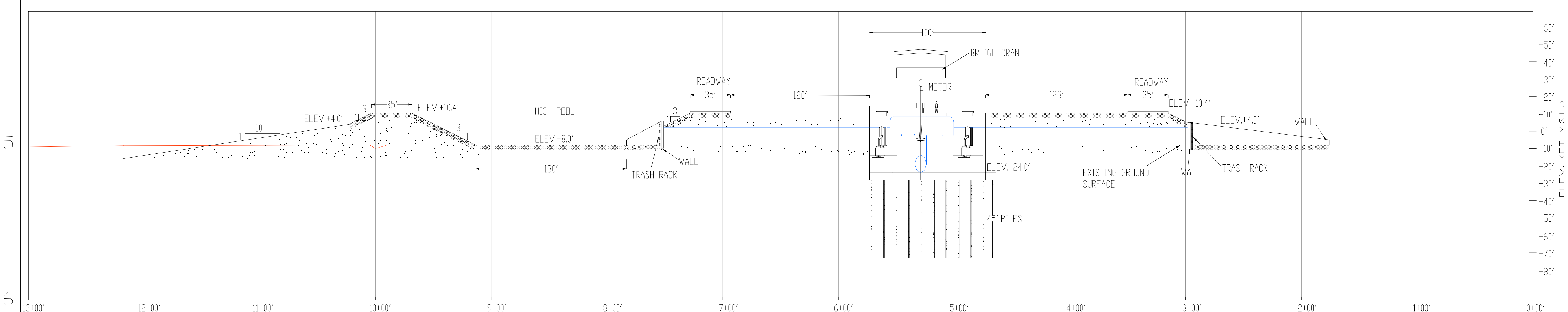


FIGURE 24

REV.	DATE	DESCRIPTION	SUB.	APP'D	DESIGNED / DRAWN / SUBMITTED	CHECKED / REVIEWED	APPROVAL RECOMMENDED: / APPROVAL RECOMMENDED: / APPROVED: DATE:		STATE OF CALIFORNIA THE RESOURCES AGENCY DEPARTMENT OF WATER RESOURCES DIVISION OF PLANNING AND LOCAL ASSISTANCE INTEGRATED STORAGE INVESTIGATIONS	IN DELTA STORAGE PROGRAM	SPEC. No. / DRAWING No. / REV. SHEET No.

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VICTORIA ISLAND
(OLD RIVER INTEGRATED FACILITY)

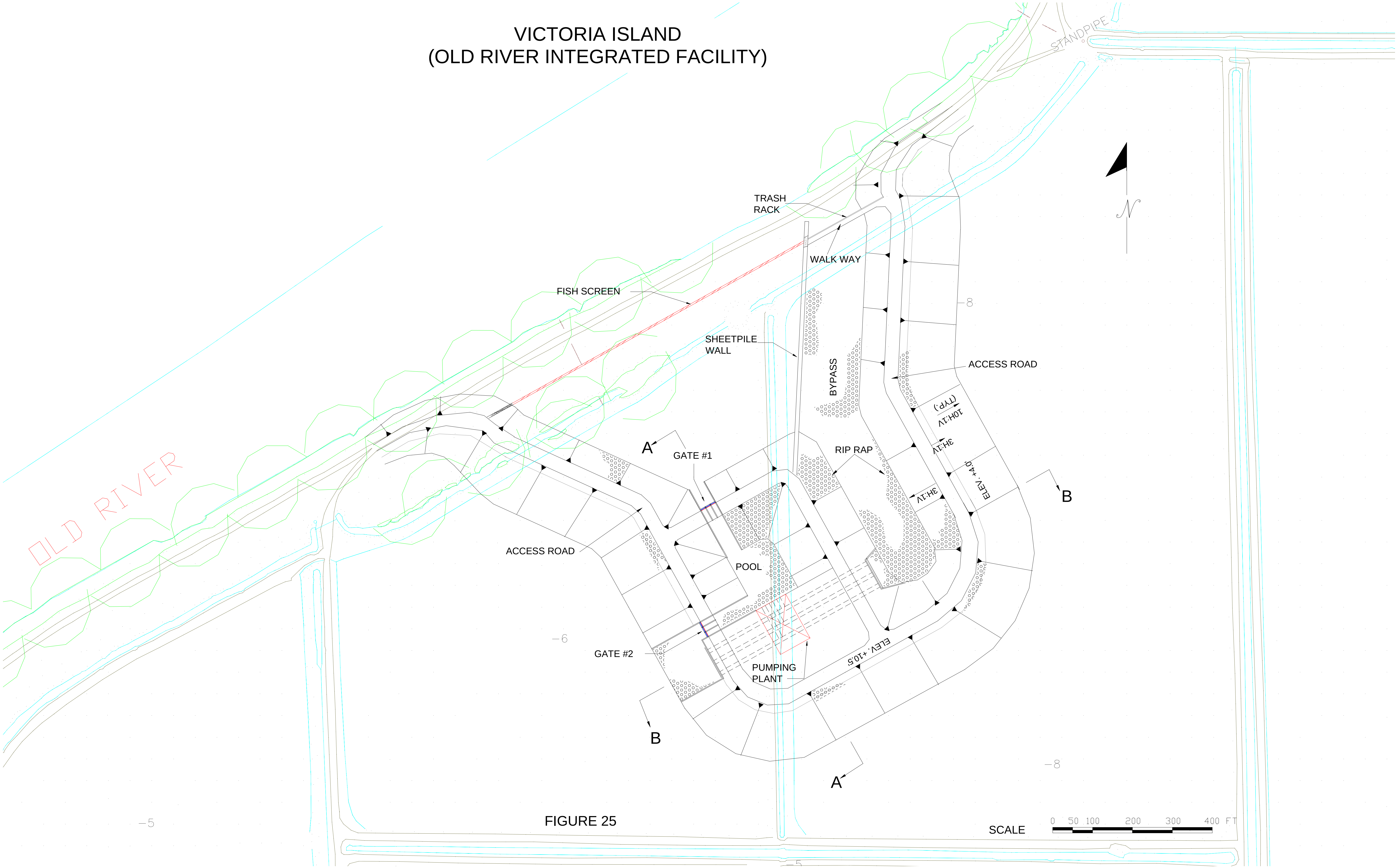
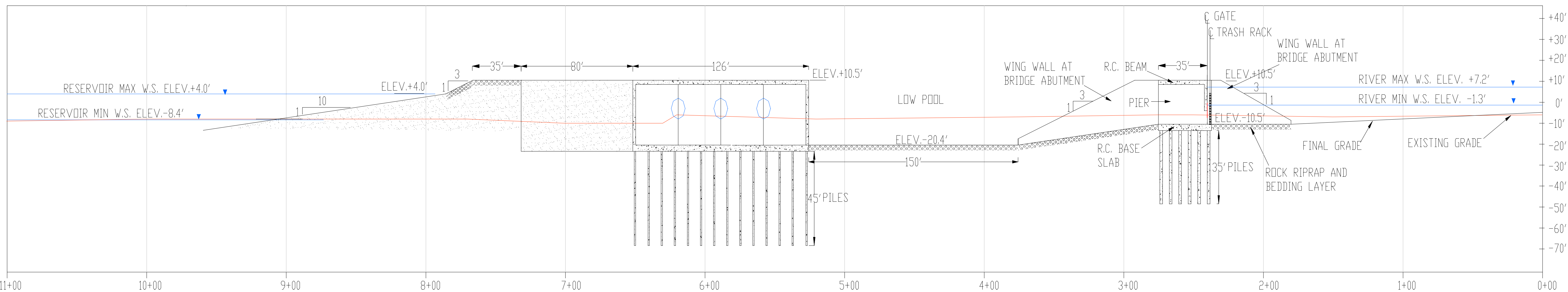


FIGURE 25

VICTORIA ISLAND
(Old River Integrated Facility)

SECTION -AA



SECTION -BB

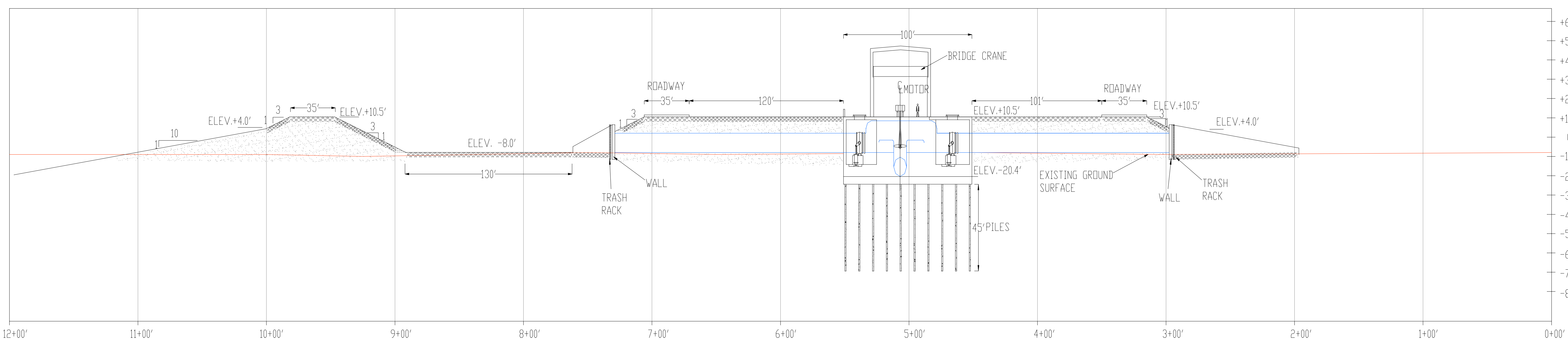


FIGURE 26

REV.	DATE	DESCRIPTION	SUB.	APP'D

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DESIGNFILESPECIFICATION

DATE
DESIGNFILESPECIFICATION

VICTORIA ISLAND
(MIDDLE RIVER INTEGRATED FACILITY)

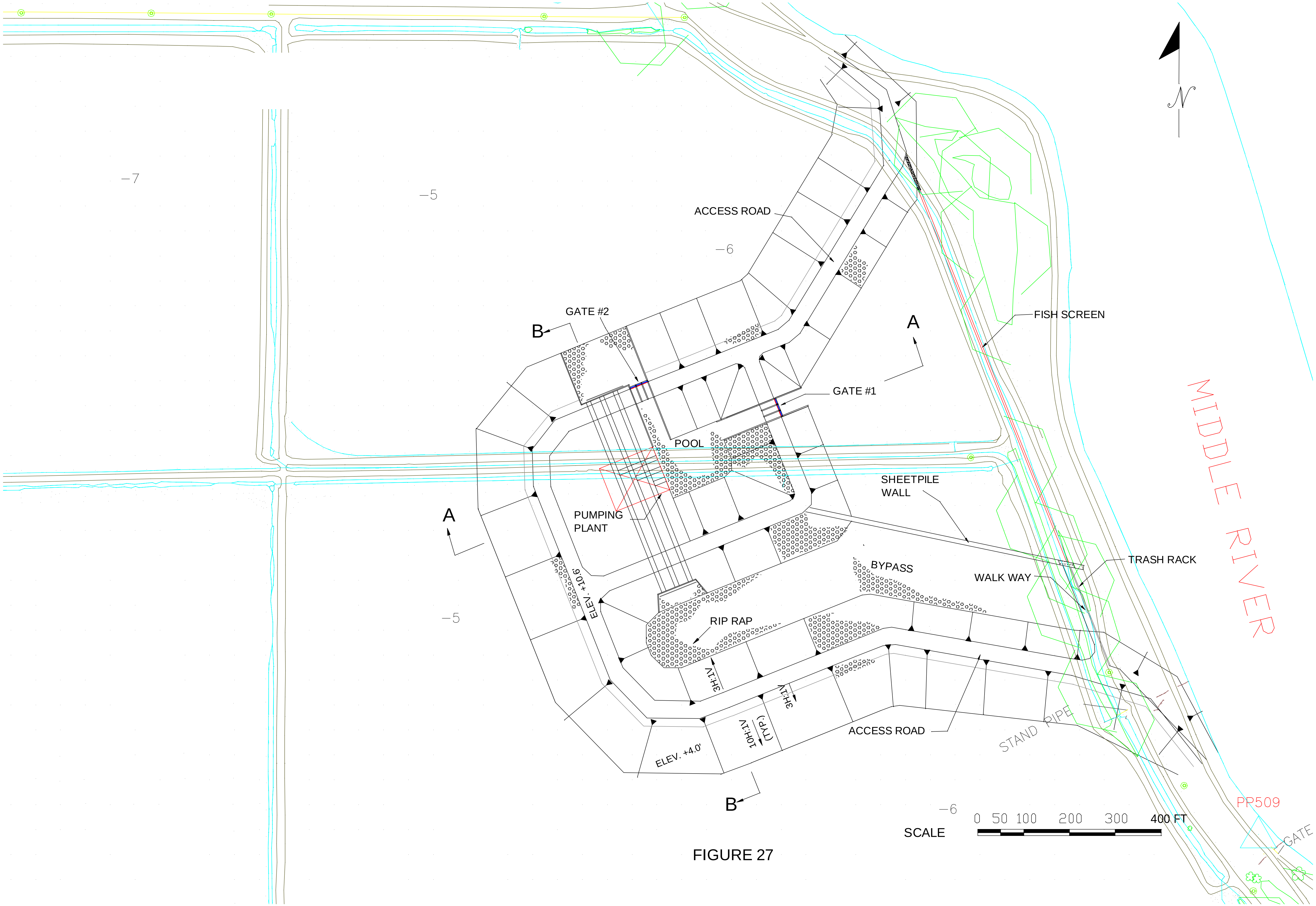
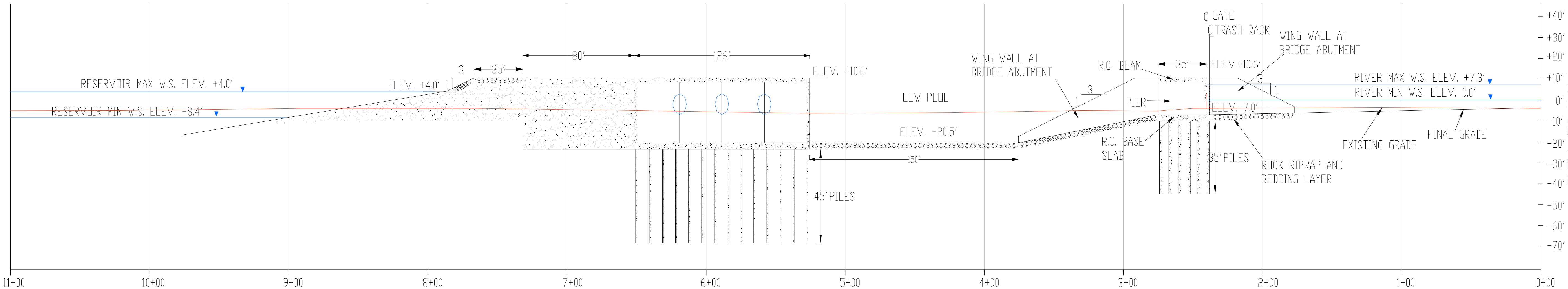


FIGURE 27

VICTORIA ISLAND
(Middle River Integrated Facility)

SECTION -AA



SECTION -BB

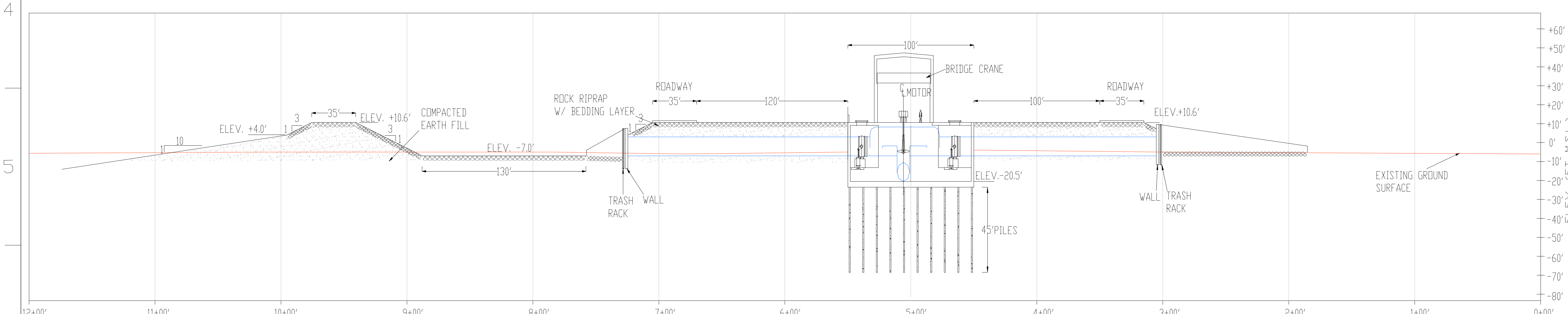


FIGURE 28

REV.	DATE	DESCRIPTION	SUB.	APP'D

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DATE
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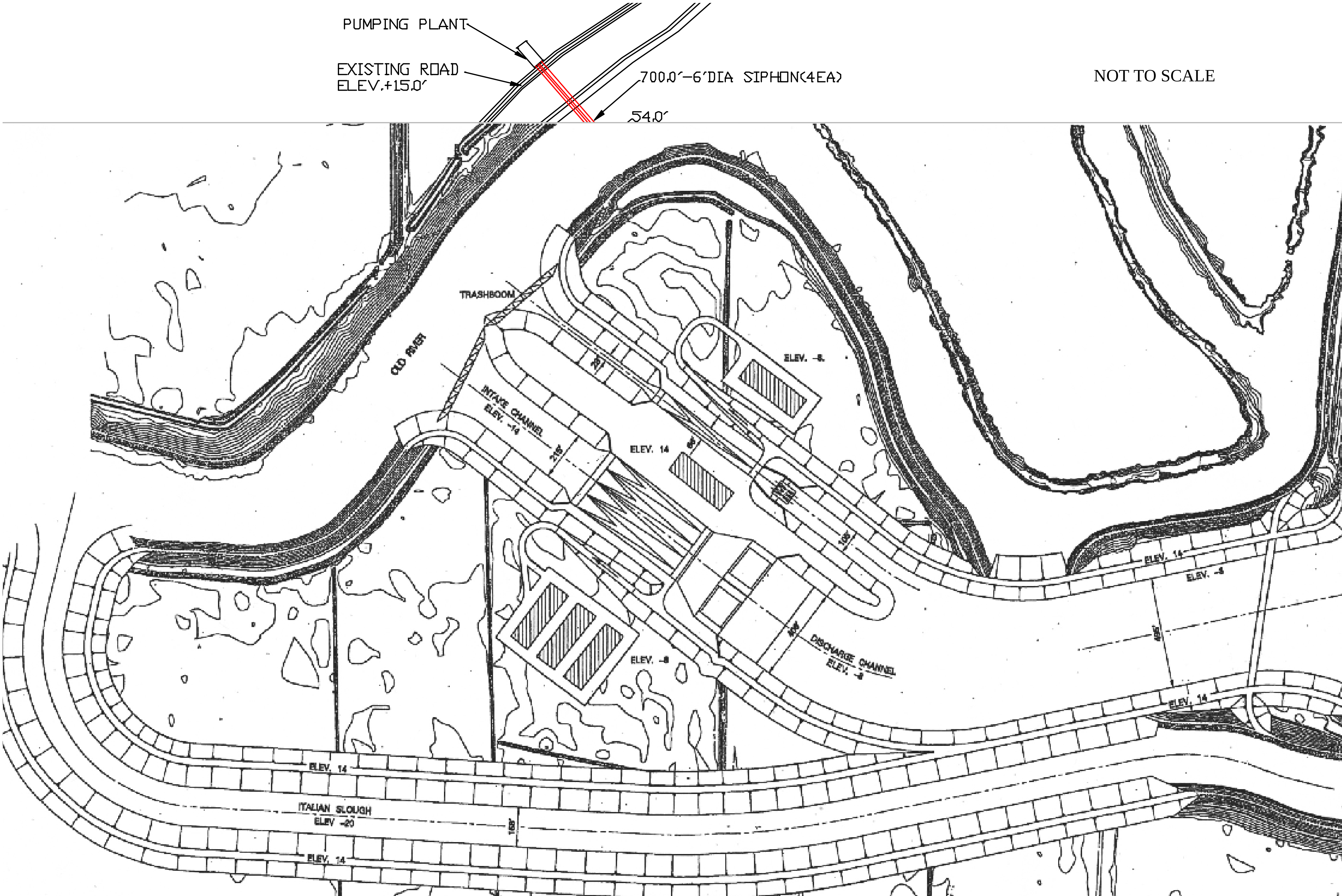
Figure 29.

SIPHON INTAKE AND CHANNEL CONVEYANCE
VICTORIA ISLAND TO NEW CLIFTON COURT INTAKE

NOT TO SCALE



TYPICAL CHANNEL CROSS-SECTION

SECTION-AA ON FIGURE 29

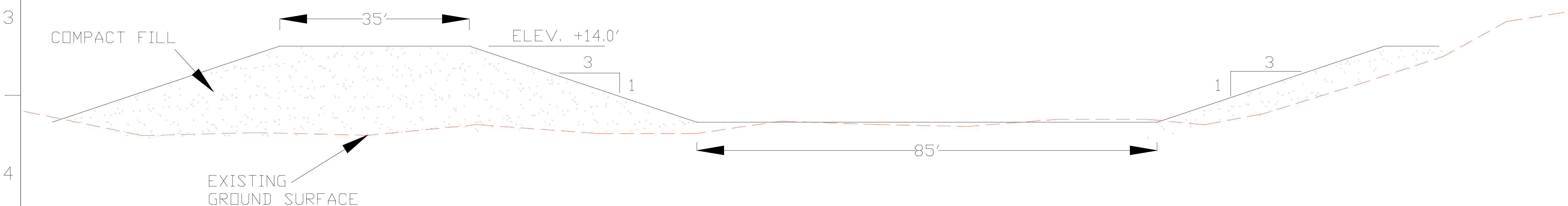


FIGURE 30

SCALE: 1''=20'

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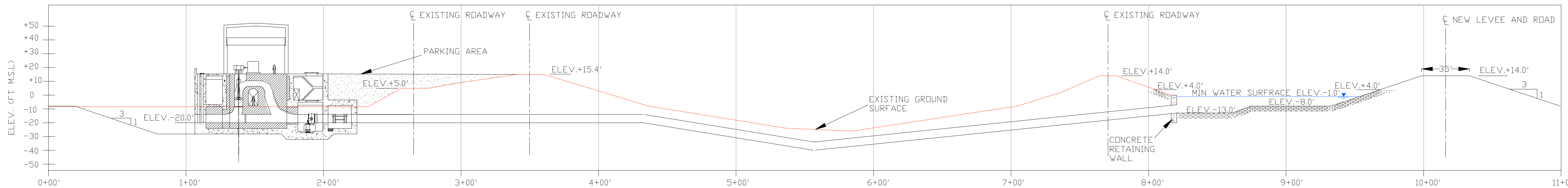
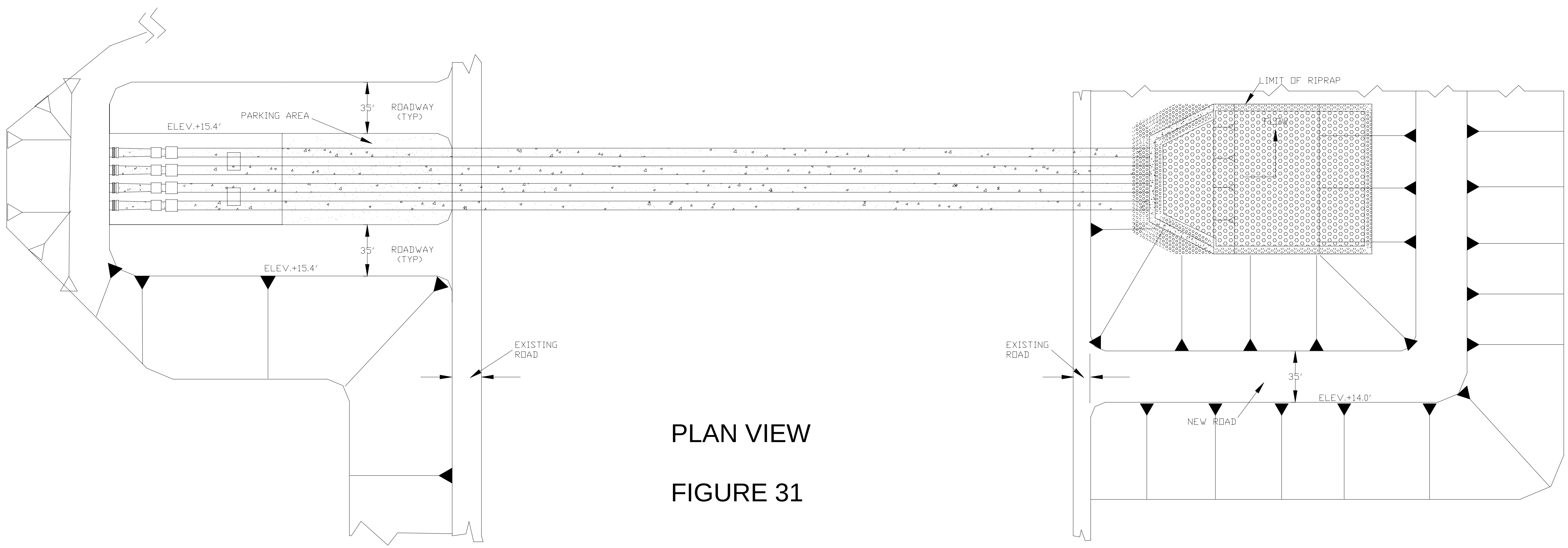
DESIGNFILESPECIFICATION

VICTORIA ISLAND
(NEW CLIFTON COURT INTAKE)





PLAN VIEW

FIGURE 31

SCALE:1"=80'

REV.	DATE	DESCRIPTION	SUB.	APP'D

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IN DELTA STORAGE PROGRAM

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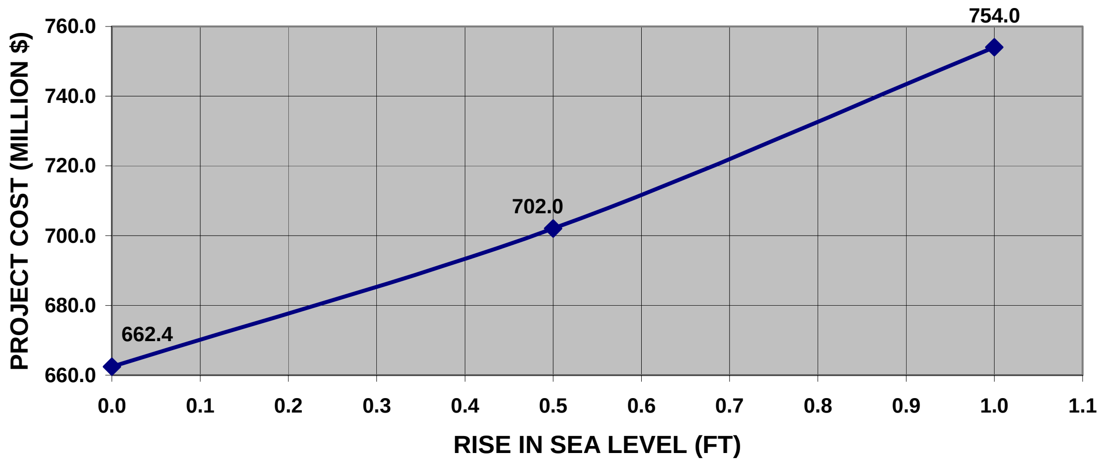
REV. SHEET No.

DESIGNFILESPECIFICATION

DATE
DESIGNFILESPECIFICATION

Figure 32. Delta Wetlands Re-engineered Project Costs due to Rise in Sea Level (6,000 cfs Max. Flow Rate)



APPENDIX C: TABLES

Table 1

Area-Capacity

WEBB TRACT

Elevation (Feet)	Cumulative Volume (Acre-Feet)	Surface Area (Acres)
		5390
4	100,664	5374
2	89,884	5359
0	79,150	5343
-2	68,486	5328
-4	57,933	5312
-6	47,551	5296
-8	37,442	5097
-10	27,837	4839
-12	18,938	4305
-14	11,084	3767
-16	4,888	2939
-18	1,230	1299
-20	146	190
-22	0	0

BACON ISLAND

Elevation (Feet)	Cumulative Volume (Acre-Feet)	Surface Area (Acres)
		5467
4	114,965	5450
2	100,527	5433
0	86,130	5415
-2	71,845	5398
-4	57,584	5380
-6	43,600	5363
-8	30,346	5345
-10	18,707	5301
-12	9,603	3736
-14	4,031	2253
-16	976	1037
-18	87	154
-20	0	0

VICTORIA ISLAND

Elevation (Feet)	Cumulative Volume (Acre-Feet)	Surface Area (Acres)
		7118
4	107,978	7102
2	93,482	7082
0	79,072	7060
-2	64,354	6995
-4	49,190	6832
-6	34,138	6565
-8	20,601	6438
-10	10,800	5840
-12	4,080	5100
-14	12	20
-16	0	0

Table 2a
Webb Tract, 100-yr Flood Height and Wave Run up

Station	Fetch Length (ft)	Wave Runup (ft)	Base Flood * (ft)
0+00	-	-	-
1+00	850	2.1	6.7
10+00	900	2.1	6.7
20+00	800	2.0	6.7
30+00	1,150	2.4	6.8
40+00	1,250	2.5	6.8
50+00	800	2.0	6.8
60+00	850	2.1	6.8
70+00	14,850	7.7	6.8
80+00	14,450	7.6	6.8
90+00	14,900	7.7	6.9
99+83	15,650	7.9	6.9
110+00	15,600	7.9	6.9
120+00	16,750	8.2	6.9
130+00	14,800	7.7	6.9
140+00	14,450	7.6	7.0
149+00	13,750	7.4	7.0
160+00	10,450	6.5	7.0
170+00	7,630	5.6	7.0
180+00	7,900	5.7	7.0
190+00	8,950	6.0	7.0
200+00	7,300	5.5	7.1
210+00	5,400	4.8	7.1
220+00	1,550	2.7	7.1
230+00	1,950	3.0	7.1
240+00	1,800	2.9	7.1
250+00	1,500	2.7	7.1
260+00	1,450	2.6	7.1
270+00	1,200	2.4	7.1
280+00	300	1.4	7.1
290+00	0	0.0	7.0
300+00	0	0.0	7.0
310+00	0	0.0	7.0
320+00	0	0.0	7.0
330+00	0	0.0	7.0
340+00	0	0.0	7.0
350+00	3,550	3.9	7.0
360+00	3,400	3.8	7.0
370+00	3,450	3.9	7.0
380+00	3,650	4.0	6.9
390+00	3,700	4.0	6.9
400+00	3,050	3.6	6.9
410+00	3,250	3.8	6.9
420+00	3,100	3.7	6.9
430+00	3,050	3.6	6.9
440+00	3,750	4.0	6.8
450+00	3,800	4.0	6.8
460+00	2,500	3.3	6.8
470+00	1,850	2.9	6.8
480+00	2,000	3.0	6.8
490+00	2,900	3.6	6.8
500+00	3,150	3.7	6.7
510+00	3,200	3.7	6.7
520+00	2,900	3.6	6.7
530+00	2,550	3.4	6.7
540+00	1,950	3.0	6.7
550+00	1,900	3.0	6.6
560+00	2,350	3.2	6.6
570+00	2,700	3.5	6.6
580+00	3,850	4.1	6.6
590+00	1,050	2.3	6.6
600+00	1,200	2.4	6.6
610+00	550	1.8	6.6
620+00	450	1.6	6.6
630+00	300	1.4	6.7
640+00	350	1.5	6.7
650+00	350	1.5	6.7
660+00	350	1.5	6.7
670+00	400	1.6	6.7
680+00	400	1.6	6.7
682+44	-	-	-

* Base Flood = 100-year Flood Elevation

Table 2b
Bacon Island, 100-yr Flood Height and Wave Run up

Station	Fetch Length (ft)	Wave Runup (ft)	Base* Flood (ft)
0+00	-	-	-
1+00	700	1.9	7.3
10+00	1,500	2.7	7.3
20+00	1,000	2.2	7.3
30+00	1,100	2.3	7.3
40+00	1,000	2.2	7.3
50+00	0	0.0	7.3
60+00	0	0.0	7.3
70+00	550	1.8	7.3
80+00	500	1.7	7.3
90+00	1,000	2.2	7.3
100+00	1,600	2.7	7.3
110+00	600	1.8	7.3
111+00	750	2.0	7.3
112+00	250	1.3	7.3
113+00	250	1.3	7.3
114+00	250	1.3	7.3
115+00	250	1.3	7.3
116+00	250	1.3	7.3
117+00	250	1.3	7.3
118+00	250	1.3	7.3
119+00	250	1.3	7.3
120+00	300	1.4	7.3
121+00	400	1.6	7.2
122+00	500	1.7	7.2
123+00	500	1.7	7.2
124+00	500	1.7	7.2
125+00	500	1.7	7.2
126+00	500	1.7	7.2
127+00	400	1.6	7.2
128+00	350	1.5	7.2
129+00	600	1.8	7.2
130+00	800	2.0	7.2
131+00	1,100	2.3	7.2
132+00	1,300	2.5	7.2
133+00	1,400	2.6	7.2
134+00	1,700	2.8	7.2
135+00	500	1.7	7.2
136+00	950	2.2	7.2
137+00	400	1.6	7.2
138+00	600	1.8	7.2
139+00	550	1.8	7.2
140+00	500	1.7	7.2
141+00	550	1.8	7.2
142+00	500	1.7	7.2
143+00	500	1.7	7.2
144+00	500	1.7	7.2
145+00	550	1.8	7.2
146+00	550	1.8	7.2
147+00	550	1.8	7.2

Station	Fetch Length (ft)	Wave Runup (ft)	Base* Flood (ft)
148+00	500	1.7	7.2
149+00	500	1.9	7.2
150+00	500	2.7	7.2
151+00	500	2.2	7.2
152+00	0	0.0	7.2
153+00	0	0.0	7.2
154+00	0	0.0	7.2
155+00	0	0.0	7.2
156+00	0	0.0	7.2
157+00	0	0.0	7.2
158+00	0	0.0	7.2
159+00	1,300	2.7	7.2
160+00	1,300	1.8	7.2
161+00	1,400	2.0	7.2
162+00	1,450	1.3	7.2
163+00	1,500	1.3	7.2
164+00	1,500	1.3	7.2
165+00	1,400	1.3	7.2
166+00	1,100	1.3	7.2
167+00	800	1.3	7.2
169+00	700	1.3	7.2
170+00	600	1.3	7.2
171+00	550	1.4	7.2
172+00	550	1.6	7.2
173+00	500	1.7	7.2
174+00	500	1.7	7.2
175+00	500	1.7	7.2
176+00	450	1.7	7.2
177+00	450	1.7	7.2
178+00	450	1.6	7.2
179+00	450	1.5	7.2
180+00	450	1.8	7.2
190+00	200	2.0	7.2
200+00	200	2.3	7.2
210+00	350	2.5	7.2
220+00	500	2.6	7.2
230+00	600	2.8	7.2
240+00	400	1.7	7.2
250+00	1,100	2.2	7.2
260+00	450	1.6	7.1
270+00	1,600	1.8	7.1
280+00	400	1.8	7.1
290+00	500	1.7	7.1
300+00	600	1.8	7.1
310+00	200	1.7	7.1
320+00	1,200	1.7	7.1
330+00	200	1.7	7.1
340+00	1,400	1.8	7.1
350+00	450	1.8	7.1
360+00	600	1.8	7.1

Station	Fetch Length (ft)	Wave Runup (ft)	Base * Flood (ft)
370+00	200	1.2	7.1
380+00	900	1.9	7.1
390+00	300	2.7	7.1
400+00	150	2.2	7.2
410+00	800	2.3	7.2
420+00	700	2.2	7.2
430+00	600	1.2	7.2
440+00	500	1.2	7.2
450+00	525	1.8	7.2
460+00	525	1.7	7.2
461+00	525	2.2	7.2
462+00	525	2.7	7.2
463+00	525	1.8	7.2
464+00	525	2.0	7.2
465+00	525	1.3	7.2
466+00	525	1.3	7.2
467+00	525	1.3	7.2
468+00	525	1.3	7.2
469+00	525	1.3	7.2
470+00	525	1.3	7.2
480+00	250	1.3	7.2
490+00	700	1.3	7.2
500+00	600	1.4	7.2
510+00	600	1.6	7.2
520+00	600	1.7	7.2
530+00	700	1.7	7.2
540+00	550	1.7	7.2
550+00	600	1.7	7.3
560+00	200	1.7	7.3
570+00	1,000	1.6	7.3
580+00	300	1.5	7.3
590+00	300	1.8	7.3
600+00	400	2.0	7.3
610+00	1,400	2.3	7.3
620+00	800	2.5	7.3
630+00	300	2.6	7.3
640+00	300	2.8	7.3
650+00	300	1.7	7.4
660+00	300	2.2	7.4
670+00	300	1.6	7.4
680+00	300	1.8	7.4
690+00	300	1.8	7.4
700+00	800	1.7	7.4
710+00	1,200	1.8	7.4
720+00	500	1.7	7.4
730+00	500	1.7	7.4
740+00	600	1.7	7.4
750+00	550	1.8	7.3
756+36	-	-	-

* Base Flood = 100-year Flood Elevation

Table 3. Delta Wetlands Assumed Borrow Requirements (from 2001 EIS, Table 3D-4)

Island	Borrow Quantity, cubic yards	Borrow Site Configuration		
		Depth, feet	Total area, acres	Average Size, acres
Bacon Island	330,000	5	41	10
Webb Tract	410,000	5	51	10
Total for Alt. 1*	4,180,000	5	405	10

* Includes interior dikes and modification of levees on Bouldin Island and Holland Tract

Table 4. URS Estimation of Wave Runup, Setup and Crest Elevation

	Bacon Island		Webb Tract	
	5:1 slope	3:1 slope	5:1 slope	3:1 slope
Wave runup without riprap (feet)	4.0	6.4	3.8	6.1
Wave runup with riprap (feet)	2.2	3.5	2.1	3.4
Reservoir setup (feet)	0.4	0.4	0.3	0.3
Crest elevation, reservoir at El. 4	6.6	7.9	6.4	7.7

Table 5. - Minimum Factors of Safety

CASE	MATERIAL PROPERTIES	PHREATIC SURFACE	MINIMUM FACTOR OF SAFETY			
			USACE levee	DWR	USBR	use
End of Construction	Unconsolidated undrained shear strength	Construction-induced excess pore pressures with high and low river elevations	1.3	1.3	1.3	1.3
Sudden Drawdown	Consolidated undrained shear strength	Rapid drawdown from normal pool to dead storage with low river elevation (use phreatic surface from steady-state seepage with surface following the island slope.	1.0	1.25	1.3	1.2
Steady-state Seepage	Consolidated drained strength	Steady-state seepage under normal pool with low river elevation	1.4*	1.5**	1.5	1.5
Post-liquefaction Stability	Based on SPT	Steady-state	1.0	1.0	1.2	1.1

* Nonproject Delta Levees per PL84-99, factor of safety is 1.25
** California DWR Delta Levees (1989b), factor of safety is 1.3

Table 6. - Webb Tract - Existing Levee Configurations

VARIABLE	AVERAGE	MAX	MIN	MEDIAN	Std. DEV.	Delta Wetlands	
						Sta. 160*	**Sta. 630***
Crest Elevation	8.5	10.7	7.3	8.3	0.7	8.3	7.5
Crest Width, feet	19.2	28.7	12.5	18.2	3.6	17.7	28
Height of Levee, feet[1]	18.3	22.1	9.2	18.5	2.6	18.8	19.5
Water Side Slope[2]	2.6	4.2	1.4	2.5	1.3	2.8	2.5
Upper Land Side Slope[3]	3.8	12.8	1.6	3.2	2.2	2.7	2.8
Lower Land Side Slope[4]	12.7	39.0	0	11.8	9.1	10.5	16.7
Thickness of Peat, feet[5]	26.8	40	10	30	8	30	10
Channel Elevation	-25	-41	-13	-25	5.9	28	25

*Specific cross section not available in CALFED data so averaged adjacent sections

[1]*Height of Levee - crest elevation minus the approximate elevation of the island at the toe.*
[2]*Water side slope - slope from crest elevation 0.*
[3]*Upper landside slope - slope from crest to first noticeable break in the slope.*
[4]*Lower landside slope - slope from first noticeable slope break to the next*
[5]*Thickness of peat taken from 1998 CALFED study which used Organic material depths from the Department of Water Resources' map entitled, "Organic Isopach Map", October 18, 1976.*

Table 7. - Bacon Island - Existing Levee Configurations

VARIABLE	AVERAGE	MAX	MIN	MEDIAN	STdev	Sta. 25*	Sta. 265*
Crest Elevation	8.2	10.9	6.6	8.0	0.6	8.1	8.1
Crest Width, feet	26.3	56.4	9.8	26.4	7.6	26.9	18.1
Height of Levee, feet[1]	16.8	25.5	7.0	17.5	3.4	15.5	19.7
Water Side Slope[2]	2.7	15.0	1	2.4	1.4	2.5	2.6
Upper Land Side Slope[3]	3.7	17.4	1	3.0	2.6	2.8	4.0
Lower Land Side Slope[4]	9.3	43.3	0	7.4	3.5	9.1	24.2
Thickness of Peat, feet[5]	14	20	10	10	5	10	15

*Specific cross section not available in CALFED data so averaged adjacent sections

[1]Height of Levee - crest elevation minus the approximate elevation of the island at the toe.
[2]Water side slope - slope from crest elevation 0.
[3]Upper landside slope - slope from crest to first noticeable break in the slope.
[4]Lower landside slope - slope from first noticeable slope break to the next
[5]Thickness of peat taken from 1998 CALFED study which used Organic material depths from the Department of Water Resources' map entitled, "Organic Isopach Map", October 18, 1976.

Table 8. - Typical Configurations for Analysis

VARIABLE	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Height, feet	10	10	10	24	24	24	16	16
Water Side Slope, H:V	3:1	2:1	2:1	3:1	2:1	2:1	2:1	2:1
Land Side Slope, H:V	4:1	4:1	4:1	4.5:1	4:1	4:1	4:1	3.5:1
Thickness of Peat, feet	10	20	40	10	20	40	20	30
New Crest Elevation	10	10	10	10	10	10	15	15

Table 9. Material Properties

Material	Weight γ, lb/ft³		Unconsolidated Undrained Strength		Consolidated Drained Strength		Consolidated Undrained Strength	
	Wet	Sat.	C lb/ft²	φ, degrees	C', lb/ft²	Φ' degrees	C', lb/ft²	Φ', degrees
New fill	110 (120)	120 [115]	0	30	0 (0) [0]	30 (34) [35]	0	30
Existing fill, sand	(110)	110	0	30	0 (0)	30 (32)	0	30
Existing fill, sand with clay and peat	(110) [105] {115}	110 {130}	0 (135)	30 (12)	0 (80) [0] {0}	30 (27) [35] {30}	0	30
Peat under dam @ centerline		70 [70] {83}	50-1500 (135) [100-300]	0 (12) [0]	50 (50) [50] {50}	28 (28) [30] {19}	100	15
Free field peat	(70)	70	250 (135) [100-300]	0 (12)	50 (50)	26 (26)	100	15
Deep sand	-	125 [125] {125}	-	-	0 [0] {0}	36 [37] {40}	0	36
Gray fat clay	-	85	200-300 [200-300]	0 [0]	0 [100]	25 [30]	100	30

() values used by URS Greiner Woodward Clyde in the July 2000 EIR Review report.
[] values used by Harding and Lawson in the 1989 study.
{} values used by State of California in the 1990 Levee Rehabilitation study

Table 10. Water Levels Used in Stability Analysis

Condition	Water Elevation (ft)		Direction of Failure Surface
	River/slough	Island	
Existing	(6)* (0)	(-16) (-16)	Island River/slough
End of construction	0 and 6 (2 and 6)	GS** (-16)	Island River/slough
Steady-state seepage	6 (6) 0 (0)	GS (-14) 4 (6)	Island River/slough
Sudden drawdown	6 (0) 0	GS (-14) (6)	Island River/slough
Seismic	(2) 0 (0)	(-14) 4 (6)	Island River/slough

* Numbers in brackets are the values used in the URS [] study.
** GS = 2 feet below ground surface

Table 11. Factors of Safety from Harding and Lawson Associates Analysis (1989)

Island Profile	Existing Condition		After Construction		Long-term	
	Slough	Island	Slough	Island	Slough	Island
Bacon #3 (Sta. 265)	1.6	1.2	1.5	1.6	1.6	1.8
Bacon #4 (Sta. 25)	2.0	1.5	1.7	2.4	1.7	2.5
Webb #7 (Sta. 160)	1.4	1.4	1.3	1.6	1.4	1.8
Webb #8 (Sta. 630)	1.5	2.0	1.4	3.1	1.4	5.3
Design Criteria	n/a	n/a	1.3	1.3	1.5	1.5

Table 12. Factors of Safety from URS Greiner Woodward Clyde Analysis (2000)

Island Profile	Existing Condition		End of Construction		Long-term		Sudden Drawdown	
	Slough	Island	Slough	Island	Slough	Island	Slough	Island
Bacon Sta. 25+00	1.48	1.23	1.48	0.9	1.33	1.63	1.33	1.07
Bacon Sta.265+00	1.49	1.21	1.49	0.86	1.23	1.64	1.23	0.98
Webb Sta.160+00	1.29	1.24	1.29	0.62	1.12	1.57	1.12	0.88
Webb Sta.630+00	1.34	1.40	1,34	0.89	1.12	1.82	1.12	1.18
Webb * Sta.630+00	-	-	-	1.22	1.12	1.71	-	1.04
Design Criteria	n/a	n/a	1.3	1.3	1.5	1.5	1.2	1.2

* new fill has 3:1 slope flattening to 10:1 at elevation -3.

Table 13. Reclamation/DWR Factors of Safety for Steady-state Condition and Sliding Towards River/Slough

<table>
<tr><th rowspan="2">Slope (H:V) above Elevation 0</th><th rowspan="2">Peat Strength free field//under dam//cohesion (phi//phi//psf)</th><th colspan="4">Factor of Safety* 10' embankment</th><th colspan="4">Factor of Safety* 18' embankment</th></tr>
<tr><th colspan="2">10' peat</th><th colspan="2">30' peat</th><th colspan="2">10' peat</th><th colspan="2">30' peat</th></tr>
<tr><td>2:1</td><td>30//0</td><td>.95</td><td>1.55</td><td colspan="2">.95</td><td colspan="2">1.24</td><td colspan="2">1.14</td></tr>
<tr><td>3:1</td><td>30//0</td><td>1.13</td><td></td><td colspan="2">1.04</td><td colspan="2">1.37</td><td colspan="2">1.19</td></tr>
<tr><td>4:1</td><td>30//0</td><td>1.33</td><td>1.54</td><td colspan="2">1.13</td><td colspan="2">1.51</td><td colspan="2">1.24</td></tr>
<tr><td>2:1</td><td>26//28//50</td><td>1.19</td><td>1.68</td><td>1.16</td><td>1.28</td><td>1.43</td><td>1.65</td><td>1.25</td><td>1.31</td></tr>
<tr><td>3:1</td><td>26//28//50</td><td>1.31</td><td>1.88</td><td>1.24</td><td>1.39</td><td>1.51</td><td>1.79</td><td>1.29</td><td>1.35</td></tr>
<tr><td>4:1</td><td>26//28//50</td><td>1.56</td><td>2.34</td><td>1.39</td><td>1.64</td><td>1.73</td><td>2.29</td><td>1.43</td><td>1.59</td></tr>
<tr><td>2:1</td><td>15//19//100</td><td colspan="2">1.2</td><td colspan="2">1.08</td><td colspan="2">1.36</td><td colspan="2">1.1</td></tr>
<tr><td>3:1</td><td>15//19//100</td><td colspan="2">1.28</td><td colspan="2">1.12</td><td colspan="2">1.43</td><td colspan="2">1.17</td></tr>
<tr><td>4:1</td><td>15//19//100</td><td colspan="2">1.46</td><td colspan="2">1.17</td><td colspan="2">1.53</td><td colspan="2">1.22</td></tr>
</table>

* Where there are two values reported, the first value is the factor of safety that takes out only a portion of the crest and the other factor of safety is for a sliding surface that includes the entire crest.

Table 14. Reclamation/DWR Factors of Safety for Steady-state Condition and Sliding Towards Island

	1	2	3	4	5	6	7	8
Height of Existing Embankment, feet	10	10	10	24	24	24	16	16
Thickness of peat, feet	10	20	40	10	20	40	20	30
New Crest Elevation	10	10	10	10	10	10	15	15
Factor of Safety	1.75	1.41	1.26	2.71	1.96	1.49	1.67	1.46

Assumes existing slope is approximately 4:1, new slope is 3:1 to elevation +4 and then 10:1, slough side slope is cut back to 4:1, and a new crest width of 35 feet, reservoir empty and river at elevation +6.

Table 15. Reclamation/DWR Factors of Safety for Post Liquefaction Condition and Sliding Towards River/Slough

Liquefied Strength, psf	**Factor of Safety* 10' embankment**		**Factor of Safety* 18' embankment**	
	10' peat	30' peat	10' peat	30' peat
100	0.93	1.11	0.91	1.04
200	1.21	1.29	1.20	1.22
400	1.58	1.40	1.70	1.43
no liquef.	1.56	1.39	1.73	1.43

* Assumed 4:1 slope on the river/slough side, water in the slough to elevation 0 , no water in the reservoir, free field peat strength assumed to be c=50 psf and φ = 26, peat under embankment strength assumed to be c=50 psf and φ = 28

Table16. Reclamation/DWR Factors of Safety for Post Liquefaction Condition and Sliding Towards Island

	1	2	3	4	5	6	7	8
Height of Existing Embankment, feet	10	10	10	24	24	24	16	16
Thickness of peat, feet	10	20	40	10	20	40	20	30
New Crest Elevation	10	10	10	10	10	10	15	15
Factor of Safety- no liquef.	1.80	1.41	1.26	2.71	1.96	1.49	1.67	1.46
Factor of Safety for 100 psf	1.21	1.0	1.02	1.23	1.06	0.98	0.98	0.92
Factor of Safety for 200 psf	1.55	1.21	1.16	1.49	1.24	1.07	1.17	1.06
Factor of Safety for 400 psf	2.16	1.43	1.26	1.97	1.56	1.26	1.49	1.32

Assumes existing slope is approximately 4:1, new slope is 3:1 to elevation 4 and then 10:1, slough side slope is cut back to 4:1, and a new crest width of 35 feet, reservoir empty and river at elevation 6.

Table 17. Values of Hydraulic Conductivity Used by URS

Material	Hydraulic conductivity Horizontal, K_x [cm/s]	Hydraulic conductivity Vertical, K_y [cm/s]
Fill (Clay with Peat and Sand; Sand; Sand and Clay; Peat)	$1x10^{-4}$	$1x10^{-5}$
Fill (Clay)	$1x10^{-6}$	$1x10^{-6}$
Peat	$1x10^{-4}$	$1x10^{-5}$
Sand	$1x10^{-3}$	$1x10^{-4}$
Channel Silt	$1x10^{-6}$	$1x10^{-6}$
Upper Clay	$1x10^{-6}$	$1x10^{-6}$
Lower Clay	$1x10^{-6}$	$1x10^{-6}$
New Fill (Sand)	$1x10^{-3}$	$1x10^{-3}$

Table 18. Estimated Peat Settlement, feet [Harding Lawson Associates]

PEAT THICKNESS (feet)	HEIGHT OF FILL (feet)		
	5	10	15
	ESTIMATED PEAT SETTLEMENT (feet)		
5	1.5	2.5	3.0
10	3.0	5.0	7.0
15	5.0	8.0	11.0
20	7.0	11.5	15.0

Table 19. Integrated Facility Gate and Valve Operation

Condition	Method	Gate #1	Gate #2	Valves on Intake/Discharge Conduit	
				Reservoir Side	River Side
Diversion	Pumped	Open	Closed	Open	Closed
Diversion	Gravity	Open	Open	Open	Closed
Release	Pumped	Closed	Open	Closed	Open
Release	Gravity	Open	Open	Open	Open

TABLE 20. Gate Design Data and Dimensions

FACILITY	DESIGN FLOW RATE	MAX. HEAD DIFFERENCE BEWEEN RIVER AND RESERVOIR	GATE OPENING (d) BASED ON		FINAL DESIGN GATE HEIGHT
			Equation 1	Equations 2 and 3	
	(cfs)	(ft)	(ft)	(ft)	(ft)
Webb Tract – San Joaquin River	1,500	21	1.9	6.5	8
Bacon Island – Middle River	1,500	22	1.8	6.5	8
Victoria Island – Middle River	1,500	18	2.0	6.5	8
Webb Tract – False River	1,500	21	1.5	5.2	7
Bacon Island _ Santa Fe Cut	1,500	22	1.5	5.2	7
Victoria Island – Old River	1,500	18	1.6	5.2	7

(Note: Number or Bays or Gate Sections = 3; Clear Width of Each Gate Section = 12 ft.)

Table 21. Low Pool and Bypass Dimensions and Settings

FACILITY	LOW POOL BOTTOM		BYPASS AT DOWNSTREAM END	
	SIZE (FT)	ELEVATION (FT)	WIDTH (FT)	ELEVATION (FT)
Webb Tract – San Joaquin River	120 X 150	-24.0	130	-15.5
Bacon Island – Middle River	120 X 150	-24.0	130	-14
Victoria Island – Middle River	120 X 150	-20.5	130	-6
Webb Tract – False River	120 X 150	-24.0	130	-14
Bacon Island – Santa Fe Cut	120 X 150	-24.0	130	-8
Victoria Island – Old River	120 X 150	-20.5	130	-8

Table 22. Total Dynamic Head and Installed Capacity of the Integrated Facilities

Facility Location	Design Flow	Head Loss in Trash Rack	Contraction loss:(flow from river to gated channel)	Misc. Minor Losses (Gate, Valve, Bend and Exit)	Pipe Friction Head Loss	Total Head Loss	Max. Water Surface Elev.- River or Resv.	Min. Water Surface Elev.-Low Pool	Max. Static Head	Total Dynamic Head	Installed Capacity Reqd.	Installed Capacity Reqd.
	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(hp)	(mw)
Webb Tract – False River	1500	0.04	0.03	1.83	0.22	2.12	6.84	-14	21	23	4,499	3.4
Bacon Island – Santa Fe Cut	1500	0.04	0.03	1.83	0.22	2.12	7.30	-14.5	22	24	4,695	3.5
Victoria Island – Old River	1500	0.04	0.03	1.83	0.22	2.12	7.24	-11	18	20	3,912	2.9
Webb Tract – San Joaquin River	1500	0.04	0.03	1.83	0.22	2.12	6.84	-14	21	23	4,499	3.4
Bacon Island – Middle River	1500	0.04	0.03	1.83	0.22	2.12	7.30	-14.5	22	24	4,695	3.5
Victoria Island – Middle River	1500	0.04	0.03	1.83	0.22	2.12	7.30	-11	18	20	3,912	2.9

Table 23. Annual Operation and Maintenance Cost

Alternative	Activity										
	Embankment Maintenance	Integrated Facilities & Fish Screen Maintenance	Pump Operations	Seepage Control System	Habitat Island Monitoring and O&M	Fisheries Mitigation & Monitoring	Invasive Weed Control on Reservoir Islands	Recreation	Cultural Resources Mitigation	Property Taxes	Total O&M Cost
Re-Engineered Delta Wetlands	$ 837,000	$ 200,000	$ 1,003,000	$ 502,000	$ 1,400,000	$ 2,800,000	$ 970,000	$ 265,000	$ 10,000	$ 346,000	$ 8,334,000
Bacon Island and Victoria Island with connection to Clifton Court	$ 874,000	$ 200,000	$ 934,000	$ 502,000	$ 1,400,000	$ 2,800,000	$ 985,000	$ 280,000	$ 10,000	$ 374,000	$ 8,358,000
Webb Tract and Victoria Island with connection to Clifton Court	$ 830,000	$ 200,000	$ 919,000	$ 502,000	$ 1,400,000	$ 2,800,000	$ 985,000	$ 280,000	$ 10,000	$ 373,000	$ 8,299,000

TABLE 24A. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COST
FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
1. RELOCATIONS					
New Transmission Lines to Pumping Plants					
Webb Tract	6	MI	$265,000	$1,590,000	
Bacon Island	6	MI	$265,000	$1,590,000	
PG&E Relocation at Bacon Island		LS		$9,200,000	
SUBTOTAL RELOCATIONS					$12,380,000
2. INTEGRATED FACILITIES					
2.A Webb Tract- San Joaquin River, (Max. diversion = 1,500					
cfs, Max. Release= 1,500 cfs)					
2.A.1 Pumping Plant(Q=1,500 cfs,TDH=23 ft,P=3.4 mw)	1	LS	$9,000,000	$9,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	21,700	CY	$10	$217,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,000	CY	$5	$25,000	
Dewatering		LS		$1,000,000	
2.A.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	845,100	CY	$8	$6,761,000	
Riprap	98,000	TON	$26	$2,548,000	2.5-foot deep riprap layer
Bedding	20,200	CY	$20	$404,000	1-foot deep bedding layer
2.A.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.A.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.A.5 Control Building	400	SF	$200	$80,000	
2.A.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.A.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
2.B Webb Tract- False River, (Max. diversion = 1,500 cfs,					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
Max. Release= 1,500 cfs)					
2.B.1 Pumping Plant(Q=1,500 cfs,TDH=23 ft,P=3.4 mw)	1	LS	$9,000,000	$9,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	18,900	CY	$10	$189,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,700	CY	$5	$29,000	
Dewatering		LS		$1,000,000	
2.B.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	995,800	CY	$8	$7,966,000	
Riprap	94,100	TON	$26	$2,447,000	2.5-foot deep riprap layer
Bedding	19,300	CY	$20	$386,000	1-foot deep bedding layer

TABLE 24A. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COST
FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
2.B.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.B.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.B.5 Control Building	400	SF	$200	$80,000	
2.B.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.B.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
2.C Bacon Island-Middle River, (Max. diversion = 1,500 cfs,					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
Max. Release= 1,500 cfs)					
2.C.1 Pumping Plant(Q=1,500 cfs,TDH=24 ft,P=3.5 mw)	1	LS	$10,000,000	$10,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	24,700	CY	$10	$247,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,900	CY	$5	$30,000	
Dewatering		LS		$1,000,000	
2.C.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	866,600	CY	$8	$6,933,000	
Riprap	92,600	TON	$26	$2,408,000	2.5-foot deep riprap layer
Bedding	19,100	CY	$20	$382,000	1-foot deep bedding layer
2.C.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.C.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.C.5 Control Building	400	SF	$200	$80,000	
2.C.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.C.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
2.D Bacon Island-Santa Fe Cut, (Max. diversion = 1,500 cfs,					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
Max. Release= 1,500 cfs)					
2.D.1 Pumping Plant(Q=1,500 cfs,TDH=24 ft,P=3.5 mw)	1	LS	$10,000,000	$10,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	38,200	CY	$10	$382,000	
Piles	4,300	LF	$40	$172,000	
Backfill	12,400	CY	$5	$62,000	
Dewatering		LS		$1,000,000	
2.D.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	608,800	CY	$8	$4,870,000	
Riprap	93,500	TON	$26	$2,431,000	2.5-foot deep riprap layer
Bedding	19,200	CY	$20	$384,000	1-foot deep bedding layer

TABLE 24A. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COST FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
2.D.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.D.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.D.5 Control Building	400	SF	$200	$80,000	
2.D.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.D.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
SUBTOTAL INTEGRATED FACILITIES					$104,733,000
3. FISH SCREENS					
3.1 Webb Tract-San Joaquin River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
3.2 Webb Tract-False River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
3.3 Bacon Island-Middle River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
3.4 Bacon Island-Santa Fe Cut: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
SUBTOTAL FISH SCREENS					$60,000,000
4. LAND ACQUISITION					
Bacon	5,450	AC	$3,000	$16,350,000	
Webb	5,374	AC	$3,000	$16,122,000	
Habitat Islands (Bouldin)	5,985	AC	$3,000	$17,955,000	
Habitat Islands (Holland)	3,129	AC	$3,000	$9,387,000	
SUBTOTAL LAND ACQUISITION					$59,814,000
5. ISLAND EMBANKMENTS					
Bacon					
Compacted Fill (includes a factor of 2.25)	4,400,000	CY	$8	$35,200,000	
Riprap - Slough side (includes a factor of 1.2)	461,770	TON	$26	$12,006,000	
Riprap - Reservoir side	260,000	TON	$26	$6,760,000	2.5-foot-deep riprap layer(quantity Source: Reclamation 4/17/02)
Bedding - Reservoir side	68,400	CY	$20	$1,368,000	1-foot-deep bedding layer
Road Base(20' x 6")	53,000	TON	$60	$3,180,000	
Clear and Grub	215	Acres	$2,000	$430,000	
Piping Protection	260,000	CY	$52	$13,520,000	
Webb					
Compacted Fill (includes a factor of 2.25)	3,800,000	CY	$8	$30,400,000	

TABLE 24A. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COST FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX

Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
Riprap - Slough side (includes a factor of 1.2)	764,800	TON	$26	$19,885,000	
Riprap - Reservoir side	260,000	TON	$26	$6,760,000	2.5-foot-deep riprap layer(quantity Source: Reclamation 4/17/02)
Bedding - Reservoir side	67,500	CY	$20	$1,350,000	1-foot-deep bedding layer
Road Base	48,000	TON	$60	$2,880,000	
Clear and Grub	210	Acres	$2,000	$420,000	
Piping Protection	200,000	CY	$52	$10,400,000	
SUBTOTAL ISLAND EMBANKMENTS					$144,559,000
6. DEMOLITION, CLEANUP AND MISCELLANEOUS					
Demolition and Cleanup		LS		$100,000	
Miscellaneous		LS		$8,000,000	
SUBTOTAL DEMOLITION, CLEANUP AND MISC.					$8,100,000
7. PERMITS		LS		$300,000	$300,000
8. SEEPAGE CONTROL SYSTEM					
Interceptor Wells	773	EA	$10,000	$7,730,000	20 gpm each, 150' apart, 0.5 HP, 6-stage turbine pump
Monitoring Wells	117	EA	$5,000	$585,000	
Electrical and Control Systems	773	EA	$3,000	$2,319,000	
SUBTOTAL SEEPAGE					$10,634,000
9. INTERIOR WORK					
Earthwork Excavation around Structures	600,000	CY	$4	$2,400,000	
SUBTOTAL INTERIOR WORK					$2,400,000
10. MITIGATION					
Habitat Islands Earthwork					
Habitat Development/Management					
Habitat Island Development and Construction, Fisheries					
Mitigation, Cultural Resources Mitigation, Project Construction					
Monitoring, Phase II Environmental Site Assessment					
SUBTOTAL MITIGATION			LS	$21,000,000	$21,000,000
SUBTOTAL				$423,920,000	$423,920,000
MOBILIZATION (5%)	1	LS		$21,196,000	
CONTINGENCIES/UNLISTED ITEMS (20%)				84,784,000	
CONTRACT COST SUBTOTAL				$ 529,900,000	
ENG., LEGAL, AND ADM. @ 25%				$ 132,475,000	
TOTAL PROJECT COST				**$ 662,375,000**	

TABLE 24B. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX)
(Slough-side slopes to be engineered to 4:1)

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
1. RELOCATIONS					
New Transmission Lines to Pumping Plants					
Webb Tract	6	MI	$265,000	$1,590,000	
Bacon Island	6	MI	$265,000	$1,590,000	
PG&E Relocation at Bacon Island		LS		$9,200,000	
SUBTOTAL RELOCATIONS					$12,380,000
2. INTEGRATED FACILITIES					
2.A Webb Tract- San Joaquin River, (Max. diversion = 1,500					
cfs, Max. Release= 1,500 cfs)					
2.A.1 Pumping Plant(Q=1,500 cfs,TDH=23 ft,P=3.4 mw)	1	LS	$9,000,000	$9,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	21,700	CY	$10	$217,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,000	CY	$5	$25,000	
Dewatering		LS		$1,000,000	
2.A.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	845,100	CY	$15	$12,677,000	
Riprap	98,000	TON	$39	$3,822,000	
Bedding	20,200	CY	$20	$404,000	
2.A.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.A.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.A.5 Control Building	400	SF	$200	$80,000	
2.A.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.A.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
2.B Webb Tract- False River, (Max. diversion = 1,500 cfs,					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
Max. Release= 1,500 cfs)					
2.B.1 Pumping Plant(Q=1,500 cfs,TDH=23 ft,P=3.4 mw)	1	LS	$9,000,000	$9,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	18,900	CY	$10	$189,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,700	CY	$5	$29,000	
Dewatering		LS		$1,000,000	
2.B.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	995,800	CY	$15	$14,937,000	
Riprap	94,100	TON	$39	$3,670,000	
Bedding	19,300	CY	$20	$386,000	
2.B.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	

TABLE 24B. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX)
(Slough-side slopes to be engineered to 4:1)

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
2.B.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.B.5 Control Building	400	SF	$200	$80,000	
2.B.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.B.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
2.C Bacon Island-Middle River, (Max. diversion = 1,500 cfs,					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
Max. Release= 1,500 cfs)					
2.C.1 Pumping Plant(Q=1,500 cfs,TDH=24 ft,P=3.5 mw)	1	LS	$10,000,000	$10,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	24,700	CY	$10	$247,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,900	CY	$5	$30,000	
Dewatering		LS		$1,000,000	
2.C.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	866,600	CY	$15	$12,999,000	
Riprap	92,600	TON	$39	$3,611,000	
Bedding	19,100	CY	$20	$382,000	
2.C.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.C.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.C.5 Control Building	400	SF	$200	$80,000	
2.C.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.C.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
2.D Bacon Island-Santa Fe Cut, (Max. diversion = 1,500 cfs,					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
Max. Release= 1,500 cfs)					
2.D.1 Pumping Plant(Q=1,500 cfs,TDH=24 ft,P=3.5 mw)	1	LS	$10,000,000	$10,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	38,200	CY	$10	$382,000	
Piles	4,300	LF	$40	$172,000	
Backfill	12,400	CY	$5	$62,000	
Dewatering		LS		$1,000,000	
2.D.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	608,800	CY	$15	$9,132,000	
Riprap	93,500	TON	$39	$3,647,000	
Bedding	19,200	CY	$20	$384,000	
2.D.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.D.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.D.5 Control Building	400	SF	$200	$80,000	
2.D.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.D.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12'x15'@6,000 lbs, 6 nos = 36,000 LBS, at gates

TABLE 24B. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX)
(Slough-side slopes to be engineered to 4:1)

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
					12'x12'@3,000 lbs, 6 nos = 18,000 LBS,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 LBS,at bypass channel outlet
SUBTOTAL INTEGRATED FACILITIES					$132,864,000
3. FISH SCREENS					
3.1 Webb Tract-San Joaquin River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
3.2 Webb Tract-False River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
3.3 Bacon Island-Middle River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
3.4 Bacon Island-Santa Fe Cut: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
SUBTOTAL FISH SCREENS					$60,000,000
4. LAND ACQUISITION					
Bacon	5,450	AC	$3,000	$16,350,000	
Webb	5,374	AC	$3,000	$16,122,000	
Habitat Islands (Bouldin)	5,985	AC	$3,000	$17,955,000	
Habitat Islands (Holland)	3,129	AC	$3,000	$9,387,000	
SUBTOTAL LAND ACQUISITION					$59,814,000
5. ISLAND EMBANKMENTS					
Bacon					
Compacted Fill (includes a factor of 2.25)	6,550,000	CY	$15	$98,250,000	
Riprap (2.5-foot thick)	575,000	TON	$39	$22,425,000	
Bedding (1.0-foot thick)	230,000	CY	$20	$4,600,000	
Road Base(20' x 6")	53,000	TON	$60	$3,180,000	
Clear and Grub	255	Acres	$2,000	$510,000	
Excavation	150,000	CY	$10	$1,500,000	
Piping Protection	260,000	CY	$52	$13,520,000	
Webb					
Compacted Fill (includes a factor of 2.25)	6,100,000	CY	$15	$91,500,000	
Riprap (2.5-ft thick)	550,000	TON	$39	$21,450,000	
Bedding (1.0-ft thick)	227,000	CY	$20	$4,540,000	
Road Base	48,000	TON	$60	$2,880,000	
Clear and Grub	250	Acres	$2,000	$500,000	
Excavation	125,000	CY	$10	$1,250,000	
Piping Protection	200,000	CY	$52	$10,400,000	
SUBTOTAL ISLAND EMBANKMENTS					$276,505,000
6. DEMOLITION, CLEANUP AND MISCELLANEOUS					

TABLE 24B. RE-ENGINEERED DELTA WETLANDS PROJECT: INTEGRATED FACILITIES ON WEBB TRACT AND BACON ISLAND - ESTIMATE OF QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL = 6,000 CFS MAX)
(Slough-side slopes to be engineered to 4:1)

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
Demolition and Cleanup		LS		$100,000	
Miscellaneous		LS		$8,000,000	
SUBTOTAL DEMOLITION, CLEANUP AND MISC.					$8,100,000
7. PERMITS		LS		$300,000	$300,000
8. SEEPAGE CONTROL SYSTEM					
Interceptor Wells	773	EA	$10,000	$7,730,000	20 gpm each, 150' apart, 0.5 HP, 6-stage turbine pump
Monitoring Wells	117	EA	$5,000	$585,000	
Electrical and Control Systems	773	EA	$3,000	$2,319,000	
SUBTOTAL SEEPAGE					$10,634,000
9. INTERIOR WORK					
Earthwork Excavation around Structures	600,000	CY	$4	$2,400,000	
SUBTOTAL INTERIOR WORK					$2,400,000
10. MITIGATION					
Habitat Islands Mitigation		LS		$21,000,000	
Reservoir Island Slough Side Slope Mitigation		LS		$100,000,000	Mitigation is expected to be between $100M and $200M
SUBTOTAL MITIGATION					$121,000,000
SUBTOTAL				$683,997,000	$683,997,000
MOBILIZATION (5%)	1	LS		$34,200,000	
CONTINGENCIES/UNLISTED ITEMS (20%)				136,799,000	
CONTRACT COST SUBTOTAL				$ 854,996,000	
ENG., LEGAL, AND ADM. @ 25%				$ 213,749,000	
TOTAL PROJECT COST				**$ 1,068,745,000**	

TABLE 25. BACON ISLAND AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
1. RELOCATIONS					
New Transmission Lines to Pumping Plants					
Victoria Island	3	MI	$265,000	$795,000	
Bacon Island	6	MI	$265,000	$1,590,000	
PG&E Relocation at Bacon Island		LS		$9,200,000	
Raising of Highway 4 in Victoria Island					
8'Concrete Pipe	4,116	LF	$700	$2,881,000	
Compacted Fill	2,000,000	CY	$8	$16,000,000	
Riprap	1,077,300	TON	$26	$28,010,000	
Bedding	165,000	CY	$20	$3,300,000	
Geotextile	420,000	SY	$2	$840,000	
Asphaltic Concrete	3,700	CY	$75	$278,000	
Aggregate Base	8,000	CY	$30	$240,000	
SUBTOTAL RELOCATIONS					$63,134,000
2. INTEGRATED FACILITIES					
2.A Victoria Island-Middle River, (Max. diversion = 1,500					
cfs, Max. Release= 1,500 cfs)					
2.A.1 Pumping Plant(Q=1,500 cfs,TDH=20 ft,P=2.9 mw)	1	LS	$8,000,000	$8,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	22,700	CY	$10	$227,000	
Piles	4,300	LF	$40	$172,000	
Backfill	8,100	CY	$5	$41,000	
Dewatering		LS		$1,000,000	
2.A.2 Structures Embankment					
Compacted Embankment (includes factor of 1.5)	430,400	CY	$8	$3,443,000	Victoria Island has relatively less peat soil depth, hence a
Riprap	88,100	TON	$26	$2,291,000	factor of 1.5 is used. For sites with relatively more peat soil
Bedding	18,100	CY	$20	$362,000	depth a factor of 2.25 is used.
2.A.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.A.4 Sheet Pile Separation Wall for Discharge Channel	25,200	SF	$15	$378,000	630 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.A.5 Control Building	400	SF	$200	$80,000	
2.A.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.A.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12' @3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 lbs,at bypass channel
					outlet
2.B Victoria Island-Old River, (Max. diversion = 1,500 cfs,					
Max. Release= 1,500 cfs)					
2.B.1 Pumping Plant(Q=1,500 cfs,TDH=20 ft,P=2.9 mw)	1		$8,000,000	$8,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	31,100	CY	$10	$311,000	
Piles	4,300	LF	$40	$172,000	
Backfill	7,100	CY	$5	$36,000	
Dewatering		LS		$1,000,000	
2.B.2 Structures Embankment					
Compacted Embankment (includes factor of 1.5)	556,500	CY	$8	$4,452,000	
Riprap	97,800	TON	$26	$2,543,000	
Bedding	20,000	CY	$20	$400,000	
2.B.3 Gated Structures - 2					

TABLE 25. BACON ISLAND AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.B.4 Sheet Pile Separation Wall for Discharge Channel	25,200	SF	$15	$378,000	630 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.B.5 Control Building	400	SF	$200	$80,000	
2.B.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.B.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12' @3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 lbs,at bypass channel
					outlet
2.C Bacon Island-Middle River, (Max. diversion = 1,500 cfs,					
Max. Release= 1,500 cfs)					
2.C.1 Pumping Plant(Q=1,500 cfs,TDH=24 ft,P=3.5 mw)	1	LS	$10,000,000	$10,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	24,700	CY	$10	$247,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,900	CY	$5	$30,000	
Dewatering		LS		$1,000,000	
2.C.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	866,600	CY	$8	$6,933,000	
Riprap	92,600	TON	$26	$2,408,000	
Bedding	19,100	CY	$20	$382,000	
2.C.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.C.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.C.5 Control Building	400	SF	$200	$80,000	
2.C.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.C.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12' @3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
2.D Bacon Island-Santa Fe Cut, (Max. diversion = 1,500 cfs,					12'x12'@4,000 lbs,10 nos=40,000 lbs,at bypass channel
Max. Release= 1,500 cfs)					outlet
2.D.1 Pumping Plant(Q=1,500 cfs,TDH=24 ft,P=3.5 mw)	1	LS	$10,000,000	$10,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	38,200	CY	$10	$382,000	
Piles	4,300	LF	$40	$172,000	
Backfill	12,400	CY	$5	$62,000	
Dewatering		LS		$1,000,000	
2.D.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	608,800	CY	$8	$4,870,000	
Riprap	93,500	TON	$26	$2,431,000	
Bedding	19,200	CY	$20	$384,000	
2.D.3 Gated Structures - 2					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.D.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.D.5 Control Building	400	SF	$200	$80,000	
2.D.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.D.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12'@3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
					12'x12' @4,000 lbs,10 nos=40,000 lbs,at bypass channel
					outlet

TABLE 25. BACON ISLAND AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
SUBTOTAL INTEGRATED FACILITIES					$95,855,000
3. CONVEYANCE FACILITIES					
- (from Victoria Island to Clifton Court Forebay)					
3A. Pumping Plant - South Side of Victoria Island					
Pumping into Siphons and Channel					
3.A.1 Pumping Plant(Q=2,000 cfs,TDH=29 ft,P=5.6 mw)	1	EA	$14,300,000	$14,300,000	
Excavation	44,000	CY	$10	$440,000	
Piles	3,800	LF	$40	$152,000	
Backfill	34,000	CY	$5	$170,000	
Dewatering		LS		$1,000,000	
3.A.2 Structures Embankment					
Compacted Embankment (includes a factor of 1.5)	131,400	CY	$8	$1,051,000	
Riprap	15,800	TON	$26	$411,000	
Bedding	3,100	CY	$20	$62,000	
3.A.3 Gated Structures - 4					
Piles	300	LF	$40	$12,000	
Structural Concrete	130	CY	$500	$65,000	
Vertical Slide gates (12x8)	4	EA	$144,000	$576,000	
3.A.4 Control Building	400	SF	$200	$80,000	
3.A.5 Electrical Installation	1	JOB	$100,000	$100,000	
3.A.6 Misc Metalwork (Trash Racks)	24,000	LB	$2	$48,000	
3B. Siphons (Under Old River and into Channel)					
Pipe (4 - 6' barrels), 4x700'=2,800', assume 3000'	3,000	LF	$800	$2,400,000	
Excavation	68,000	CY	$10	$680,000	
Backfill	55,500	CY	$5	$278,000	
Riprap	7,000	TON	$26	$182,000	
Bedding	1,400	CY	$20	$28,000	
Cofferdam Fill	73,750	CY	$30	$2,213,000	
Impervious Membrane	10,000	SY	$2	$20,000	
3C. Channel (conveyance from Siphons to Clifton Court FB)					
Compacted Fill (includes a factor of 1.5)	674,000	CY	$8	$5,392,000	
Riprap	221,600	TON	$26	$5,762,000	
Bedding	43,800	CY	$20	$876,000	
Outlet Culvert-CMP 12.5' dia-170' longx6 culverts	1,020	LF	$600	$612,000	
SUBTOTAL PUMPING PLANT, SIPHON and CHANNEL					$36,910,000
4. FISH SCREENS					
4.1 Webb Tract-San Joaquin River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
4.2 Webb Tract-False River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
4.3 Bacon Island-Middle River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
4.4 Bacon Island-Santa Fe Cut: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
SUBTOTAL FISH SCREENS					$60,000,000
5. LAND ACQUISITION					
Bacon	5,450	AC	$3,000	$16,350,000	
Victoria	7,102	AC	$3,000	$21,306,000	
Habitat Islands (Bouldin)	5,985	AC	$3,000	$17,955,000	

TABLE 25. BACON ISLAND AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
Habitat Islands (Holland)	3,129	AC	$3,000	$9,387,000	
SUBTOTAL LAND ACQUISITION					$64,998,000
6. ISLAND EMBANKMENTS					
6A. Bacon					
Compacted Fill (Includes a factor of 2.25)	4,400,000	CY	$8	$35,200,000	
Riprap - Slough Side (includes a factor of 1.2)	461,770	TON	$26	$12,006,000	
Riprap-Reservoir Side	260,000	TON	$26	$6,760,000	2.5-foot-deep riprap layer
Riprap Bedding-Reservoir Side	68,400	CY	$20	$1,368,000	1-foot-deep bedding layer
Road Base(20' x 6")	53,000	TON	$60	$3,180,000	
Clear and Grub	215	Acre	$2,000	$430,000	
Piping Protection	260,000	CY	$52	$13,520,000	
6B. Victoria					
Compacted Fill (Includes a factor of 1.5)	3,216,600	CY	$8	$25,733,000	Net volume, after subtracting island-side riprap and bedding
Riprap - Slough Side (includes a factor of 1.2)	818,400	TON	$26	$21,278,000	
Riprap-Reservoir Side	539,600	TON	$26	$14,030,000	2.5-foot-deep riprap layer
Riprap Bedding-Island Side	106,600	CY	$20	$2,132,000	1-foot-deep bedding layer
Road Base	55,000	TON	$60	$3,300,000	
Clear and Grub	215	Acre	$2,000	$430,000	
Piping Protection	253,000	CY	$52	$13,156,000	
SUBTOTAL ISLAND EMBANKMENTS					$152,523,000
7. DEMOLITION, CLEANUP AND MISCELLANEOUS					
Demolition and Cleanup		LS		$100,000	
Miscellaneous		LS		$8,000,000	$8,100,000
SUBTOTAL DEMOLITION, CLEANUP AND MISC.					
8. PERMITS		LS		$300,000	$300,000
9. SEEPAGE CONTROL SYSTEM				$5,000,000	
Bacon		LS			
Victoria		LS			
SUBTOTAL SEEPAGE					$5,000,000
10. INTERIOR WORK					
Earthwork Excavation around Structures	600,000	CY	$4	$2,400,000	
SUBTOTAL INTERIOR WORK					$2,400,000
11. MITIGATION					
Habitat Islands Earthwork					
Habitat Development/Management					
Habitat Island Development and Construction, Fisheries					
Mitigation, Cultural Resources Mitigation, Project Construction					
Monitoring, Phase II Environmental Site Assessment			LS	$21,000,000	
SUBTOTAL MITIGATION					$21,000,000
SUBTOTAL				**$510,220,000**	$510,220,000
MOBILIZATION (5%)	1	LS		$25,511,000	
CONTINGENCIES/UNLISTED ITEMS (20%)				102,044,000	
CONTRACT COST SUBTOTAL				$ 637,775,000	
ENG., LEGAL, AND ADM. @ 25%				$ 159,444,000	
TOTAL PROJECT COST				**$ 797,219,000**	

TABLE 26. WEBB TRACT AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX)
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
1. RELOCATIONS					
New Transmission Lines to Pumping Plants					
Victoria Island	3	MI	$265,000	$795,000	
Webb Tract	6	MI	$265,000	$1,590,000	
Raising of Highway 4 in Victoria Island					
Hwy 4 Raising					
8'Concrete Pipe	4,116	LF	$700	$2,881,000	
Compacted Fill	2,000,000	CY	$8	$16,000,000	
Riprap	1,077,300	TON	$26	$28,010,000	
Bedding	165,000	CY	$20	$3,300,000	
Geotextile	420,000	SY	$2	$840,000	
Asphaltic Concrete	3,700	CY	$75	$278,000	
Aggregate Base	8,000	CY	$30	$240,000	
SUBTOTAL RELOCATIONS					$53,934,000
2. INTEGRATED FACILITIES					
2.A Victoria Island-Middle River, (Max. diversion = 1,500					
cfs, Max. Release= 1,500 cfs)					
2.A.1 Pumping Plant(Q=1,500 cfs,TDH=20 ft,P=2.9 mw)	1	LS	$8,000,000	$8,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	22,700	CY	$10	$227,000	
Piles	4,300	LF	$40	$172,000	
Backfill	8,100	CY	$5	$41,000	
Dewatering		LS		$1,000,000	
2.A.2 Structures Embankment					
Compacted Embankment (includes factor of 1.5)	430,400	CY	$8	$3,443,000	
Riprap	88,100	TON	$26	$2,291,000	
Bedding	18,100	CY	$20	$362,000	
2.A.3 Gated Structures - 4		EA			
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.A.4 Sheet Pile Separation Wall for Discharge Channel	25,200	SF	$15	$378,000	630 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.A.5 Control Building	400	SF	$200	$80,000	
2.A.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.A.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12' @3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 lbs,at bypass channel
					outlet
2.B Victoria Island-Old River, (Max. diversion = 1,500 cfs,					
Max. Release= 1,500 cfs)					
2.B.1 Pumping Plant(Q=1,500 cfs,TDH=20 ft,P=2.9 mw)	1		$8,000,000	$8,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	31,100	CY	$10	$311,000	
Piles	4,300	LF	$40	$172,000	
Backfill	7,100	CY	$5	$36,000	
Dewatering		LS		$1,000,000	
2.B.2 Structures Embankment					

TABLE 26. WEBB TRACT AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX)
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
Compacted Embankment (includes factor of 1.5)	556,500	CY	$8	$4,452,000	
Riprap	97,800	TON	$26	$2,543,000	
Bedding	20,000	CY	$20	$400,000	
2.B.3 Gated Structures - 4					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.B.4 Sheet Pile Separation Wall for Discharge Channel	25,200	SF	$15	$378,000	630 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.B.5 Control Building	400	SF	$200	$80,000	
2.B.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.B.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12' @3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 lbs,at bypass channel
					outlet
2.C Webb Tract-San Joaquin River,(Max.diversion=1,500 cfs,					
Max. Release= 1,500 cfs)					
2.C.1 Pumping Plant(Q=1,500 cfs,TDH=23 ft,P=3.4 mw)	1	LS	$9,000,000	$9,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	21,700	CY	$10	$217,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,000	CY	$5	$25,000	
Dewatering		LS		$1,000,000	
2.C.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	845,100	CY	$8	$6,761,000	
Riprap	98,000	TON	$26	$2,548,000	
Bedding	20,200	CY	$20	$404,000	
2.C.3 Gated Structures - 4					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.C.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.C.5 Control Building	400	SF	$200	$80,000	
2.C.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.C.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12' @3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 lbs,at bypass channel
					outlet
2.D Webb Tract-False River (Max. diversion = 1,500 cfs,					
Max. Release= 1,500 cfs)					
2.D.1 Pumping Plant(Q=1,500 cfs,TDH=23 ft,P=3.4 mw)	1	LS	$9,000,000	$9,000,000	
Intake/Discharge Conduits and Miscellaneous		LS	$2,000,000	$2,000,000	
Excavation	18,900	CY	$10	$189,000	
Piles	4,300	LF	$40	$172,000	
Backfill	5,700	CY	$5	$29,000	
Dewatering		LS		$1,000,000	
2.D.2 Structures Embankment					
Compacted Embankment (includes factor of 2.25)	995,800	CY	$8	$7,966,000	
Riprap	94,100	TON	$26	$2,447,000	
Bedding	19,300	CY	$20	$386,000	

TABLE 26. WEBB TRACT AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX)
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
2.D.3 Gated Structures - 4					
Piles	3,000	LF	$40	$120,000	
Structural Concrete	4,000	CY	$500	$2,000,000	
Vertical Slide gates (12x8) - 6 nos per facility	6	EA	$144,000	$864,000	
2.D.4 Sheet Pile Separation Wall for Discharge Channel	25,600	SF	$15	$384,000	640 ft long x 20 ft high with 20 ft in-ground sheet pile wall
2.D.5 Control Building	400	SF	$200	$80,000	
2.D.6 Electrical Installation	1	JOB	$100,000	$100,000	
2.D.7 Misc Metalwork (Trash Racks)	94,000	LB	$2	$188,000	12' x 15' @ 6,000 lbs, 6 nos=36,000 lbs, at gates
					12'x12' @3,000 lbs,6 nos=18,000 lbs,at conduit intake/outlet
					12'x12'@4,000 lbs,10 nos=40,000 lbs,at bypass channel
					outlet
SUBTOTAL INTEGRATED FACILITIES					$96,698,000
3. CONVEYANCE FACILITIES					
- (from Victoria Island to Clifton Court Forebay)					
3A. Pumping Plant - South Side of Victoria Island					
Pumping into Siphons and Channel					
3.A.1 Pumping Plant(Q=2,000 cfs,TDH=29 ft,P=5.6 mw)	1	EA	$14,300,000	$14,300,000	
Excavation	44,000	CY	$10	$440,000	
Piles	3,800	LF	$40	$152,000	
Backfill	34,000	CY	$5	$170,000	
Dewatering		LS		$1,000,000	
3.A.2 Structures Embankment					
Compacted Embankment (includes factor of 1.5)	131,400	CY	$8	$1,051,000	
Riprap	15,800	TON	$26	$411,000	
Bedding	3,100	CY	$20	$62,000	
3.A.3 Gated Structures - 4					
Piles	300	LF	$40	$12,000	
Structural Concrete	130	CY	$500	$65,000	
Vertical Slide gates (12x8)	4	EA	$144,000	$576,000	
3.A.4 Control Building	400	SF	$200	$80,000	
3.A.5 Electrical Installation	1	JOB	$100,000	$100,000	
3.A.6 Misc Metalwork (Trash Racks)	24,000	LB	$2	$48,000	
3B. Siphons (Under Old River and into Channel)					
Pipe (4 - 6' barrels), 4x700'=2,800', assume 3000'	3,000	LF	$800	$2,400,000	
Excavation	68,000	CY	$10	$680,000	
Backfill	55,500	CY	$5	$278,000	
Riprap	7,000	TON	$26	$182,000	
Bedding	1,400	CY	$20	$28,000	
Cofferdam Fill	73,750	CY	$30	$2,213,000	
Impervious Membrane	10,000	SY	$2	$20,000	
3C. Channel (conveyance from Siphons to Clifton Court FB)					
Compacted Fill (Incl. Factor=1.5)	674,000	CY	$8	$5,392,000	
Riprap	221,600	TON	$26	$5,762,000	
Bedding	43,800	CY	$20	$876,000	
Outlet Culvert-CMP 12.5' dia-170' longx6 culverts	1,020	LF	$600	$612,000	
SUBTOTAL PUMPING PLANT, SIPHON and CHANNEL					$36,910,000

TABLE 26. WEBB TRACT AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX)
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
4. FISH SCREENS					
4.1 Webb Tract-San Joaquin River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
4.2 Webb Tract-False River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
4.3 Bacon Island-Middle River: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
4.4 Bacon Island-Santa Fe Cut: Screen, Deck, Sill,	1,500	CFS	$10,000	$15,000,000	
Mounting Hardware and Foundation Piles					
SUBTOTAL FISH SCREENS					$60,000,000
5. LAND ACQUISITION					
Webb	5,374	AC	$3,000	$16,122,000	
Victoria	7,102	AC	$3,000	$21,306,000	
Habitat Islands (Bouldin)	5,985	AC	$3,000	$17,955,000	
Habitat Islands (Holland)	3,129	AC	$3,000	$9,387,000	
SUBTOTAL LAND ACQUISITION					$64,770,000
6. ISLAND EMBANKMENTS					
6A. Webb					
Compacted Fill (Includes a factor of 2.25)	3,800,000	CY	$8	$30,400,000	
Riprap - Slough Side (includes a factor of 1.2)	764,800	TON	$26	$19,885,000	
Riprap-Reservoir Side	260,000	TON	$26	$6,760,000	2.5-foot-deep riprap layer (quantity source:Reclamation 4/17/02)
Bedding-Reservoir Side	67,500	CY	$20	$1,350,000	1-foot-deep bedding layer
Road Base(20' x 6")	48,000	TON	$60	$2,880,000	
Clear and Grub	210	Acres	$2,000	$420,000	
Piping Protection	200,000	CY	$52	$10,400,000	
6B. Victoria					
Compacted Fill (Includes a factor of 1.5)	3,216,600	CY	$8	$25,733,000	Net volume, after subtracting island-side riprap and bedding
Riprap - Slough Side (includes a factor of 1.2)	818,400	TON	$26	$21,278,000	
Riprap-Reservoir Side	539,600	TON	$26	$14,030,000	2.5-foot-deep riprap layer
Riprap Bedding-Island Side	106,600	CY	$20	$2,132,000	1-foot-deep bedding layer
Road Base	55,000	TON	$60	$3,300,000	
Clear and Grub	215	Acre	$2,000	$430,000	
Piping Protection	253,000	CY	$52	$13,156,000	
SUBTOTAL ISLAND EMBANKMENTS					$152,154,000
7. DEMOLITION, CLEANUP AND MISCELLANEOUS					
Demolition and Cleanup		LS		$100,000	
Miscellaneous		LS		$8,000,000	$8,100,000
8. PERMITS		LS		$300,000	$300,000
9. SEEPAGE CONTROL SYSTEM					
Webb		LS			
Victoria		LS		$5,000,000	
SUBTOTAL SEEPAGE					$5,000,000

TABLE 26. WEBB TRACT AND VICTORIA ISLAND OPTION WITH CONNECTION TO CLIFTON COURT FOREBAY
ESTIMATED QUANTITIES AND COSTS FOR 1,500 CFS DIVERSION/RELEASE AT EACH INTEGRATED FACILITY (TOTAL=6,000 CFS MAX)
Assumes Slough Side Slopes will be modified to 3:1.

Item	Total Quantity	Units	Unit Price	Pricing	Notes & Comments
10. INTERIOR WORK					
Earthwork Excavation around Structures	600,000	CY	$4	$2,400,000	
SUBTOTAL INTERIOR WORK					$2,400,000
11. MITIGATION					
Habitat Islands Earthwork					
Habitat Development/Management					
Habitat Island Development and Construction, Fisheries					
Mitigation, Cultural Resources Mitigation, Project Construction					
Monitoring, Phase II Environmental Site Assessment		LS		$21,000,000	
SUBTOTAL MITIGATION					$21,000,000
SUBTOTAL				**$501,266,000**	$501,266,000
MOBILIZATION (5%)	1	LS		$25,063,000	
CONTINGENCIES/UNLISTED ITEMS (20%)				100,253,000	
CONTRACT COST SUBTOTAL				$ 626,582,000	
ENG., LEGAL, AND ADM. @ 25%				$ 156,646,000	
TOTAL PROJECT COST				**$ 783,228,000**	

APPENDIX D

EQUATIONS USED IN PUMPING & FLOW COMPUTATIONS, RE-ENGINEERED DW PROJECT

EQUATIONS USED IN PUMPING & FLOW COMPUTATIONS, RE-ENGINEERED DW PROJECT

- Discharge equation :

$$Q = CA\sqrt{2gh} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

Where,
Q = the design flow rate in cfs
C = discharge coefficient (a conservative value of C=0.6, Ref. 36)
A = Area of the gate opening in square ft
h = head available at the gate in ft
g = acceleration of gravity in feet per second (fps)

- Average velocity at the gate is given by

$$V = \frac{Q}{Wxd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(2)$$

Where,
Q = the design flow rate in cfs
W = clear gate width (ft)
d = depth of gate opening (ft)
A = W x d

- Velocity at the pump intake is given under the design flow rate Q by

$$V = \frac{Q}{(N)\frac{\Pi}{4}D^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(3)$$

Where,
N = number of intake pipes (pumps), and
D = diameter of intake pipes (ft).

- Manning's Equation (Ref. 36)

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(4)$$

Where,
Q = the design flow rate in cfs
A = Area of the gate opening in square ft
R = hydraulic radius of the flow ft
S = slope

$$n = \frac{R^{1/6}}{23.85 + 21.95\log(R/D_{50})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5).$$

(Ref. 33)

Where,
D_{50} is the mean rock size.

- Froude Number of flow (Ref. 34)

$$F1 = \frac{V1}{\sqrt{gy1}} \quad \text{..............................(6)}$$

Where,
V1 = flow velocity (just downstream of a gate)
y1 = flow depth prior to the jump (just downstream of a gate)
g = acceleration of gravity in feet per second (fps)

- The sequent depth is calculated by the formula (Ref. 34):

$$y2 = 0.5y1(\sqrt{8F1^2 + 1} - 1) \quad \text{.................................(7)}$$

Where,
y2 = sequent depth following a hydraulic jump,
y1 = flow depth prior to the jump (just downstream of a gate)
F1 = Froude number of the flow (just downstream of a gate)

- Design flow rate for the pumps:

$$q = Q/N \quad \text{...(8)}$$

Where,
N = number of intake pumps, and
Q = the design flow rate in cfs

'q' was 500 cfs for all facilities

- Bernoulli's equation to calculate the total dynamic head (TDH) on the pump unit (Ref. 35).

$$TDH = W.S.elev_{River} - W.S.elev_{\mathrm{Re}\,servoir} + H_{Loss} \quad \text{...................(9)}$$

$$TDH = W.S.elev_{River} - W.S.elev_{\mathrm{Re}\,servoir} + h_{fs} + h_{con} + h_{tr} + h_{en} + h_{ben} + h_{pipef} + h_{val} + h_{exit} \quad \text{........(10)}$$

Where,
h_{fs} = Head loss in Fish Screen,
h_{con} = convergence loss from river to gate,
h_{tr} = head loss in Trash Rack,
h_{en} = entrance loss at pipe inlet,
h_{ben} = bending loss in the pipe,
h_{pipef} = pipe friction loss,
h_{val} = valve losses,

h_{exit} = exit loss.

- Required installation pump capacity was calculated as follows:

$$hp = \gamma q h \Big/ 550\eta \quad \text{.......................................(11)}$$

$$kw = 0.746 hp \quad \text{.......................................(12)}$$

Where,

hp = pump horse power,
γ = water unit weight, 62.4 lbs./c.ft,
q = design flow, cfs
h = Total dynamic head, ft
η = combined pump and motor efficiency, 87%
kw = kilowatt.

- Head Loss in Fish Screen, h_{fs}:

Head loss in the Fish Screen was assumed to be zero since the velocity at the fish screen is only 0.2 fps which makes the velocity head ($V^2/2g$) very low.

- Head Loss during convergence from river to the gate and reservoir to gate, h_{con}:

Head loss coefficient of 0.5 was used for a square enterance (Ref. 34)

$$h_{con} = 0.5\frac{V^2}{2g} \quad \text{.......................................(13)}$$

- Head Loss in the Trash Rack, h_{tr} (Ref. 35):

$$h_{tr} = K_t \frac{Vn^2}{2g} \quad \text{.......................................(14)}$$

Where,
Kt = trash rack loss coefficient (Ref. 35),

$$K_t = 1.45 - 0.45\left(\frac{a_n}{a_g}\right) - \left(\frac{a_n}{a_g}\right)^2 \quad \text{.................................(15)}$$

Where,
a_n=area through the trash rack bars,
a_g=gross area of trash rack and supports,
V_n=velocity through the net trash rack area.

- Entrance Loss due to sudden contraction at pipe inlet, h_{en} (Ref. 34):

$$h_{en} = 0.31\frac{V^2}{2g} \quad \text{.......................................(16)}$$

- Bending Loss in pipe, h_{ben} (Ref. 37):

$$h_{ben} = 0.4\frac{V^2}{2g} \quad \ldots\ldots(17)$$

- Head Loss due to pipe friction, h_{pipef} (Ref. 37):

$$h_{pipef} = \frac{4.66n^2Q^2L}{D^{16/3}} \quad \ldots\ldots(18)$$

- Valve Loss, h_{val} (Ref. 37):

$$h_{val} = 0.2\frac{V^2}{2g} \quad \ldots\ldots(19)$$

- Exit Loss, h_{exit}:

$$h_{exit} = 1.0\frac{V^2}{2g} \quad \ldots\ldots(20)$$

APPENDIX E

SLOPE STABILITY ANALYSIS DONE BY USBR

IN-DELTA STORAGE PROGRAM
ENGINEERING INVESTIGATIONS
DRAFT REPORT
November 19, 2001

Engineer Review Board
December 11-13, 2001

Plots from SlopeW Analysis
done by US Bureau of Reclamation

Table 3. Material Properties

Material	Weight γ, lb/ft³		Unconsolidated Undrained Strength		Consolidated Drained Strength		Consolidated Undrained Strength	
	Wet	Sat.	c lb/ft²	φ, degrees	c', lb/ft²	φ', degrees	c', lb/ft²	φ', degrees
New fill	110 (120)	120 [115]	0	30	0 (0) [0]	30 (34) [35]	0	30
Existing fill, sand	(110)	110	0	30	0 (0)	30 (32)	0	30
Existing fill, sand with clay and peat	(110) [105] {115}	110 {130}	0 (135)	30 (12)	0 (80) [0] {0}	30 (27) [35] {30}	0	30
Peat under dam @ centerline		70 [70] {83}	50-1500 (135) [100-300]	0 (12) [0]	50 (50) [50] {50}	28 (28) [30] {19}	100	15
Free field peat	(70)	70	250 (135) [100-300]	0 (12)	50 (50)	26 (26)	100	15
Deep sand	-	125 [125] {125}	-	-	0 [0] {0}	36 [37] {40}	0	36
Gray fat clay	-	85	200-300 [200-300]	0 [0]	0 [100]	25 [30]	100	30

() values used by URS Greiner Woodward Clyde in the 2000 report.
[] values used by Harding and Lawson in the 1989 study.
{} values used by State of California in the 1990 Levee Rehabilitation study

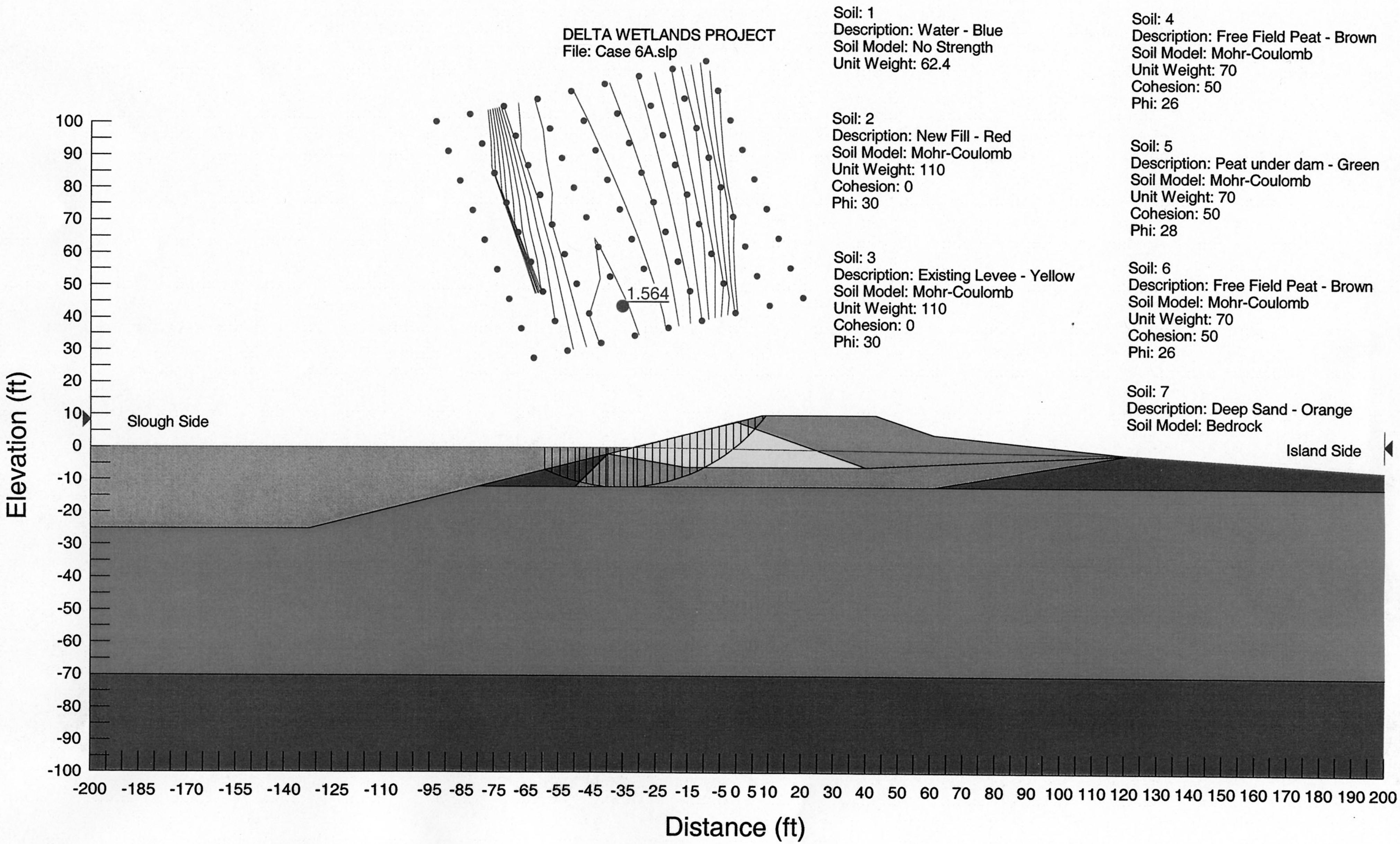
b. Steady-state Condition with Sliding Towards River/slough. - The analyses done for DW and confirmed in this analysis (as shown on Table x), indicate that a potential sliding failure into the river/sloughs exist for the steady-state condition. The factors of safety are below both the recommended dam and levee design criteria. This potential problem primarily exists where the channel is deep. The embankments with the existing slopes and a full reservoir have the potential to slide into the channels, which could cause unacceptable environmental damage, damage to floating structures, damage to adjacent levees, potential loss of life, and require expensive dredging to clean up. Loss of the reservoir may not occur because of the width of the embankments. The proposed modifications to the embankments by DW do not include modifying the river/slough side slopes. The costs for repairs and clean up of the slide mass would need to be accounted for in the overall cost of the DW proposed project.

DWR/BOR recommends that the slopes of the embankments on the river/slough side be flattened during initial construction to increase the factor of safety against a sliding failure. Careful construction control will be required to minimize environmental impact. Flattening of the slope will require the centerline of the embankments to be shifted towards the island side and increase overall fill quantities. However, material excavated can be used in the construction of the new fill. The required slopes needed to increase the factor of safety to above the design criteria will vary depending upon depth of channel, thickness of peat, strength of peat, and height of embankments. Table x shows the DWR/BOR analysis for this loading condition with two variations of embankment height and peat thickness and varying peat strengths. Based on this analysis, it is recommended that a 4:1 slope be required on the river/slough side. This slope was assumed to be an average of what will actually be required with some slopes being steeper and some needing to be flatter. During final design more specific analyses should be done to determine actual slopes needed based on additional topographic data.

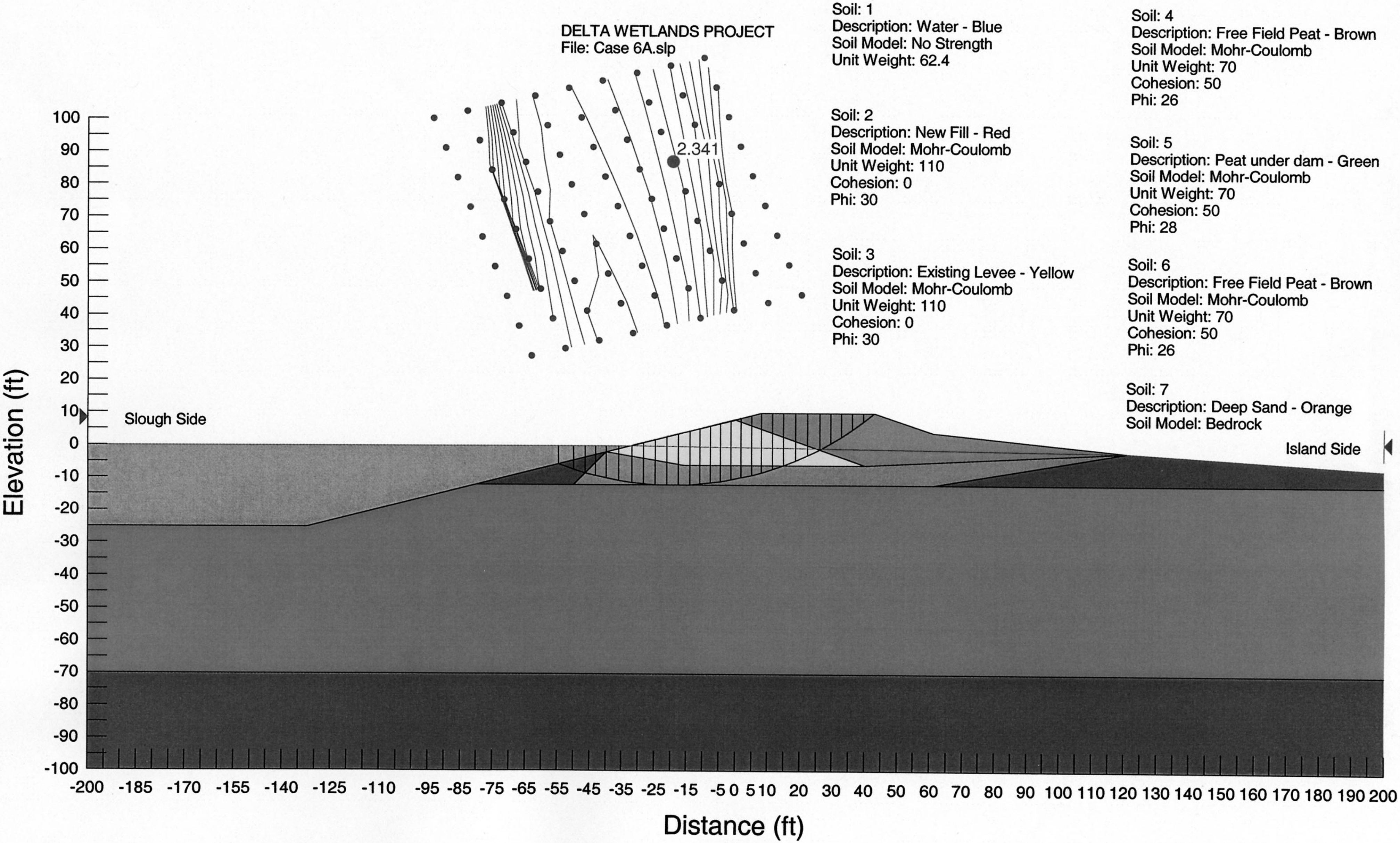
Table x. DWR/BOR Factors of Safety for Steady-state Condition and Sliding Towards River/Slough

	Slope (H:V) above Elevation 0	Peat Strength free field//under dam//cohesion (phi//phi//psf)	Factor of Safety* 10' embankment				Factor of Safety* 18' embankment			
			10' peat		30' peat		10' peat		30' peat	
			A	A	B	B	C	C	D	D
1	2:1	30//0	.95	1.55	.95		1.24		1.14	
2	3:1	30//0	1.13		1.04		1.37		1.19	
3	4:1	30//0	1.33	1.54	1.13		1.51		1.24	
4	2:1	26//28//50	1.19	1.68	1.16	1.28	1.43	1.65	1.25	1.31
5	3:1	26//28//50	1.31	1.88	1.24	1.39	1.51	1.79	1.29	1.35
Pl. # 6	4:1	26//28//50	1.56	2.34	1.39	1.64	1.73	2.29	1.43	1.59
7	2:1	15//19//100	1.2		1.08		1.36		1.1	
8	3:1	15//19//100	1.28		1.12		1.43		1.17	
9	4:1	15//19//100	1.46		1.17		1.53		1.22	

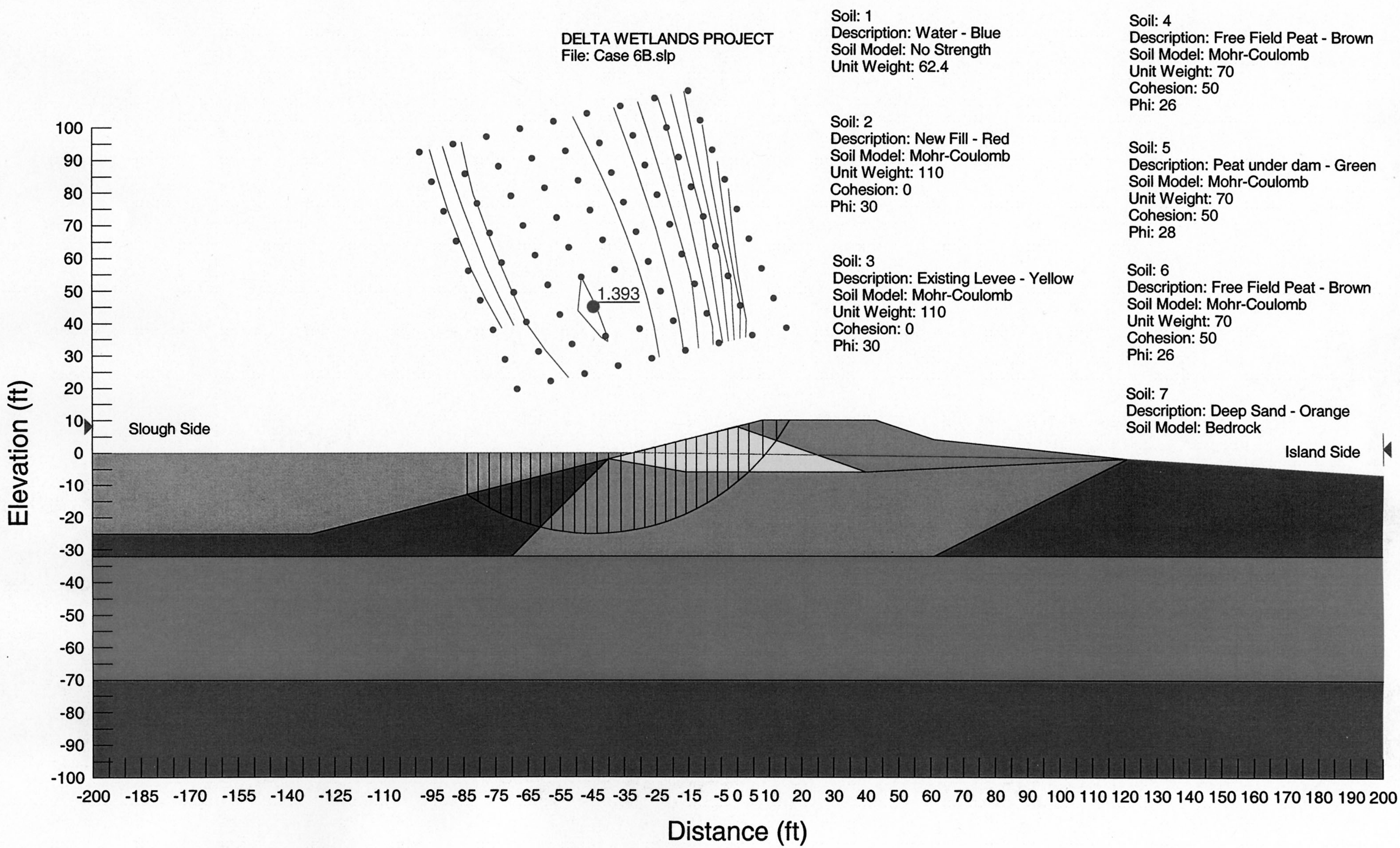
* Where there are two values reported, the first value is the factor of safety that takes out only a portion of the crest and the other factor of safety is for a sliding surface that includes the entire crest.

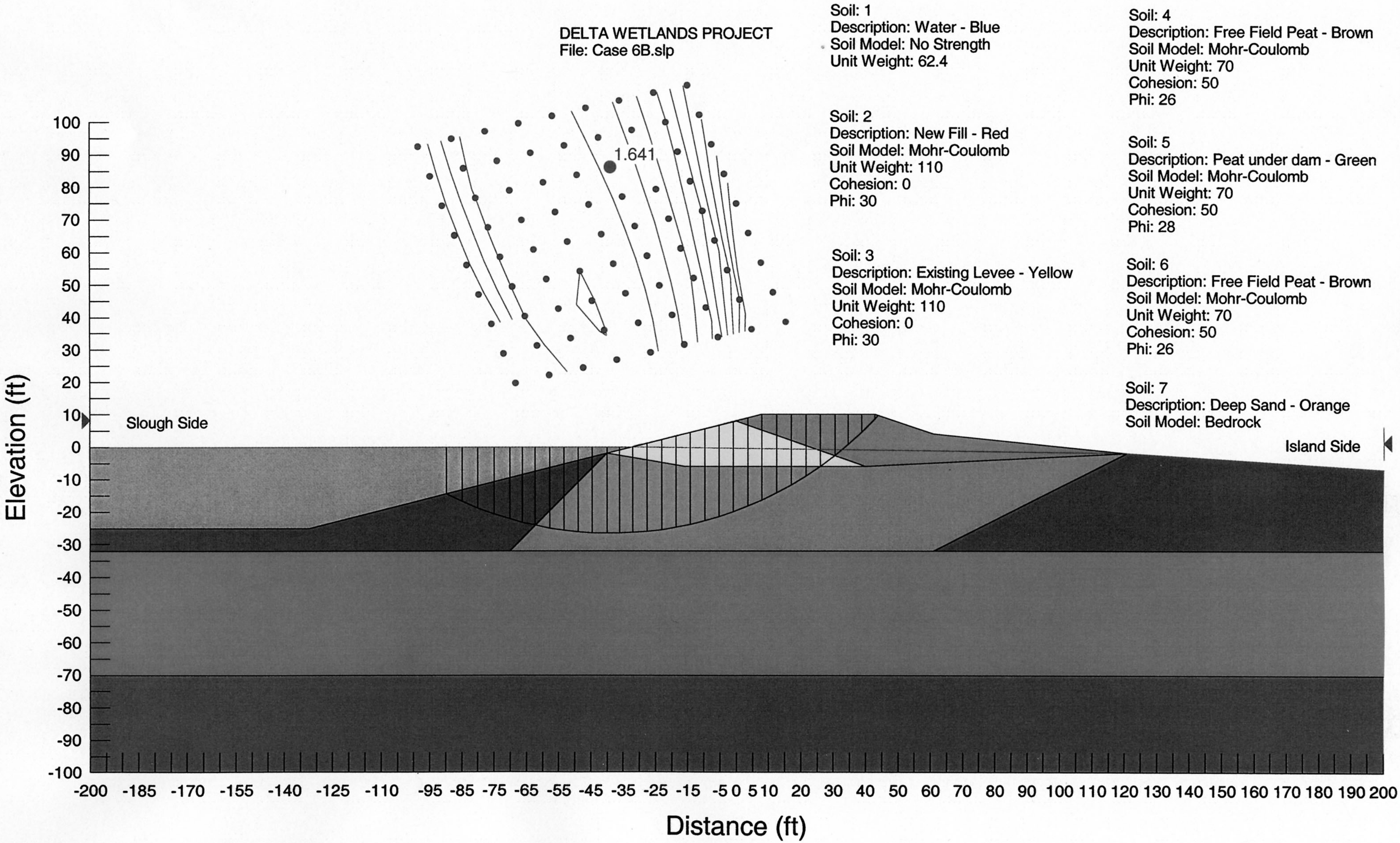


DELTA WETLANDS PROJECT
File: Case 6A.slp
1.564
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Distance (ft)

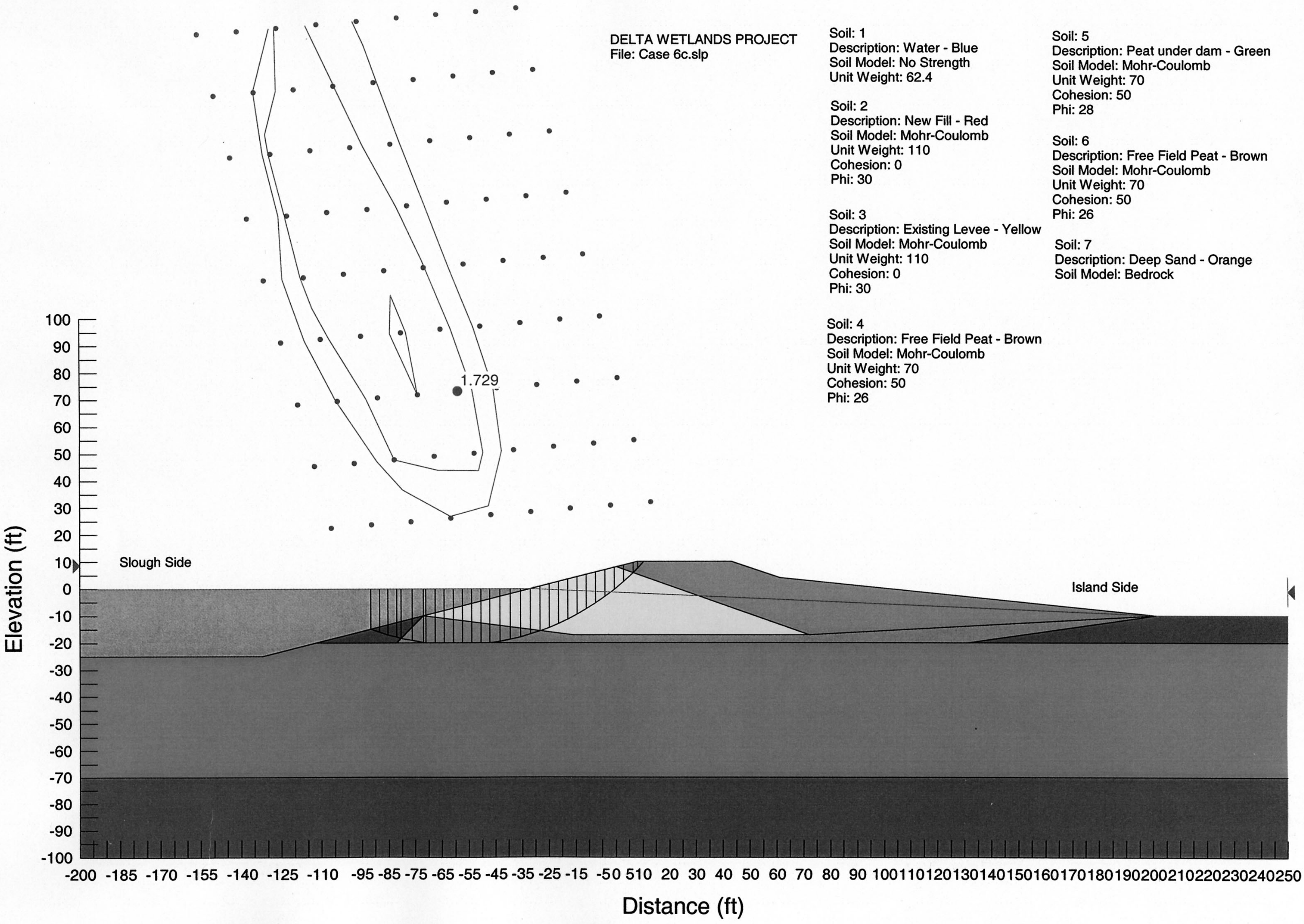


DELTA WETLANDS PROJECT
File: Case 6A.slp
2.341
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
Slough Side
Island Side
Elevation (ft)
Distance (ft)

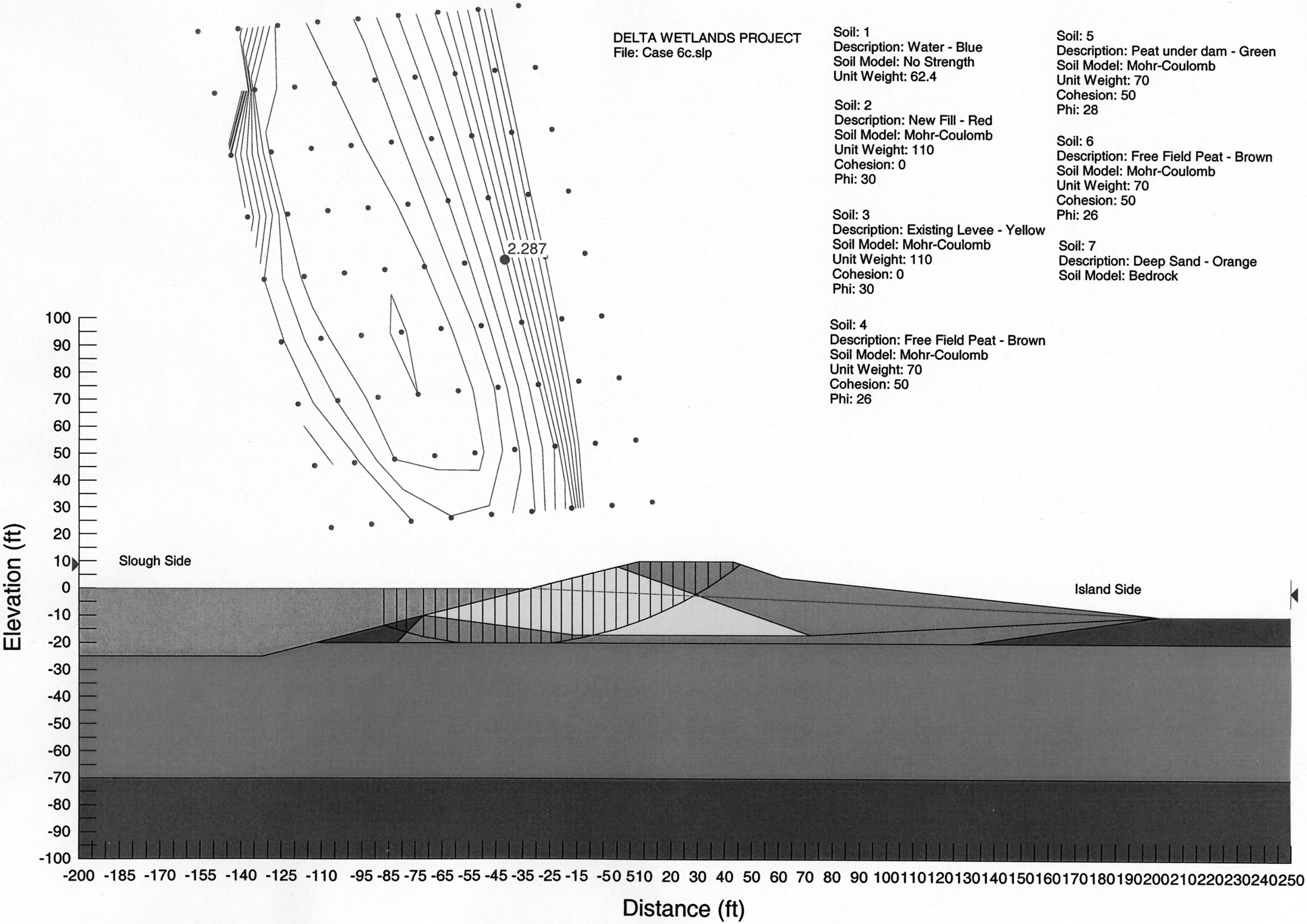

DELTA WETLANDS PROJECT
File: Case 6B.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
1.393
Slough Side
Island Side
Elevation (ft)
Distance (ft)

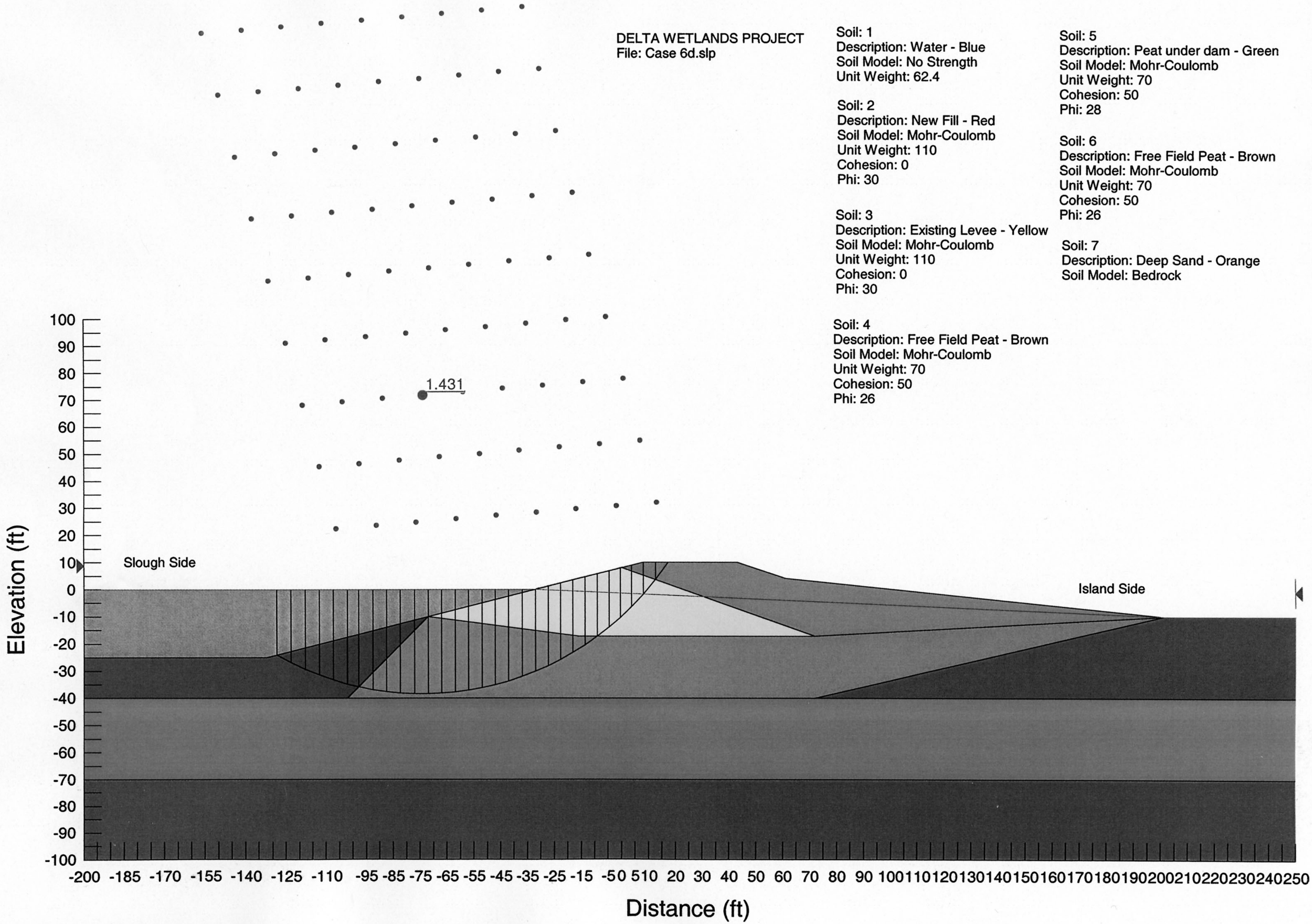


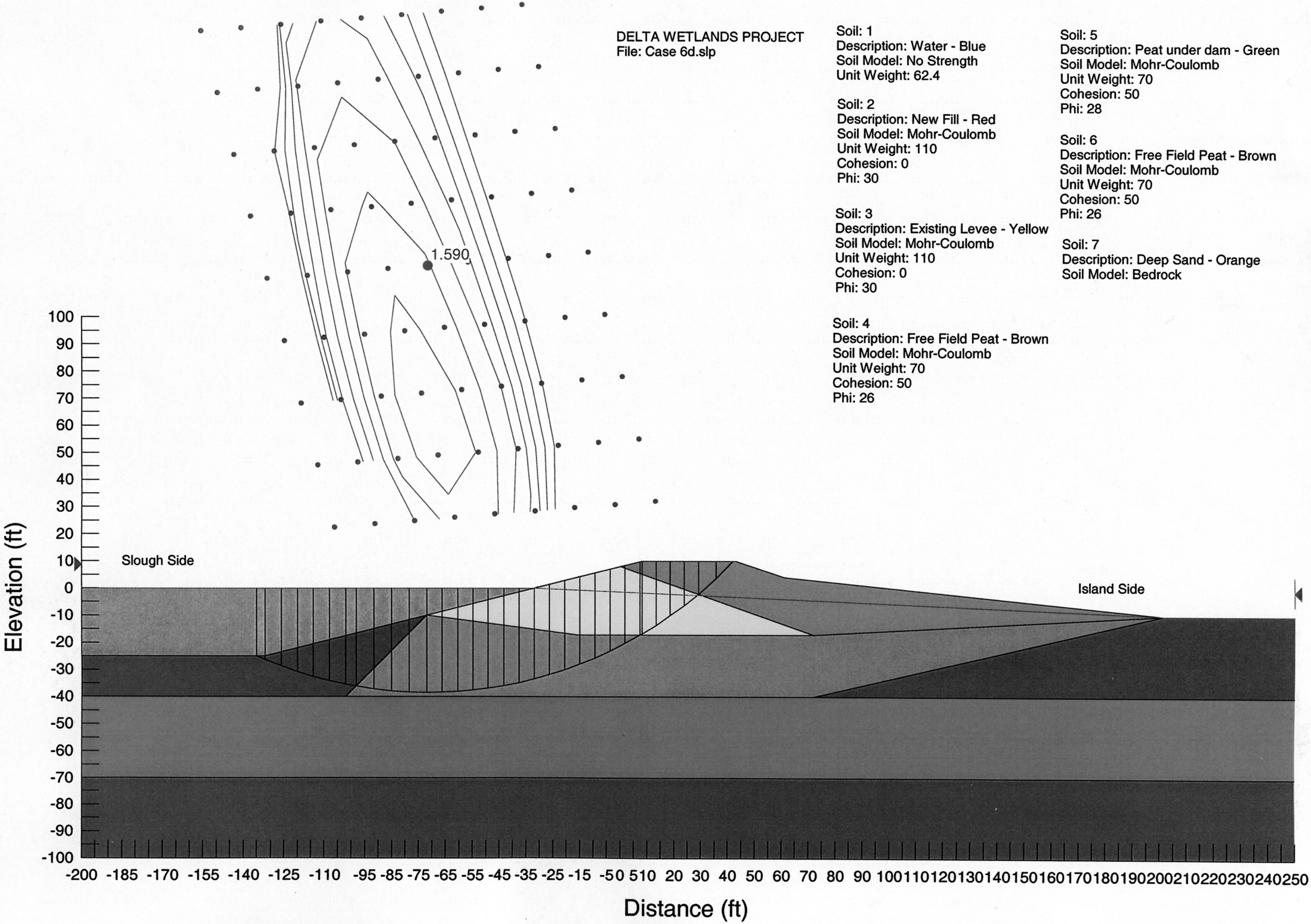
DELTA WETLANDS PROJECT
File: Case 6B.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
1.641
Slough Side
Island Side
Elevation (ft)
Distance (ft)



DELTA WETLANDS PROJECT
File: Case 6c.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
1.729
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250
Distance (ft)


DELTA WETLANDS PROJECT
File: Case 6c.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
2.287
Slough Side
Island Side
Elevation (ft)
Distance (ft)


DELTA WETLANDS PROJECT
File: Case 6d.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
1.431
Slough Side
Island Side
Elevation (ft)
Distance (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250



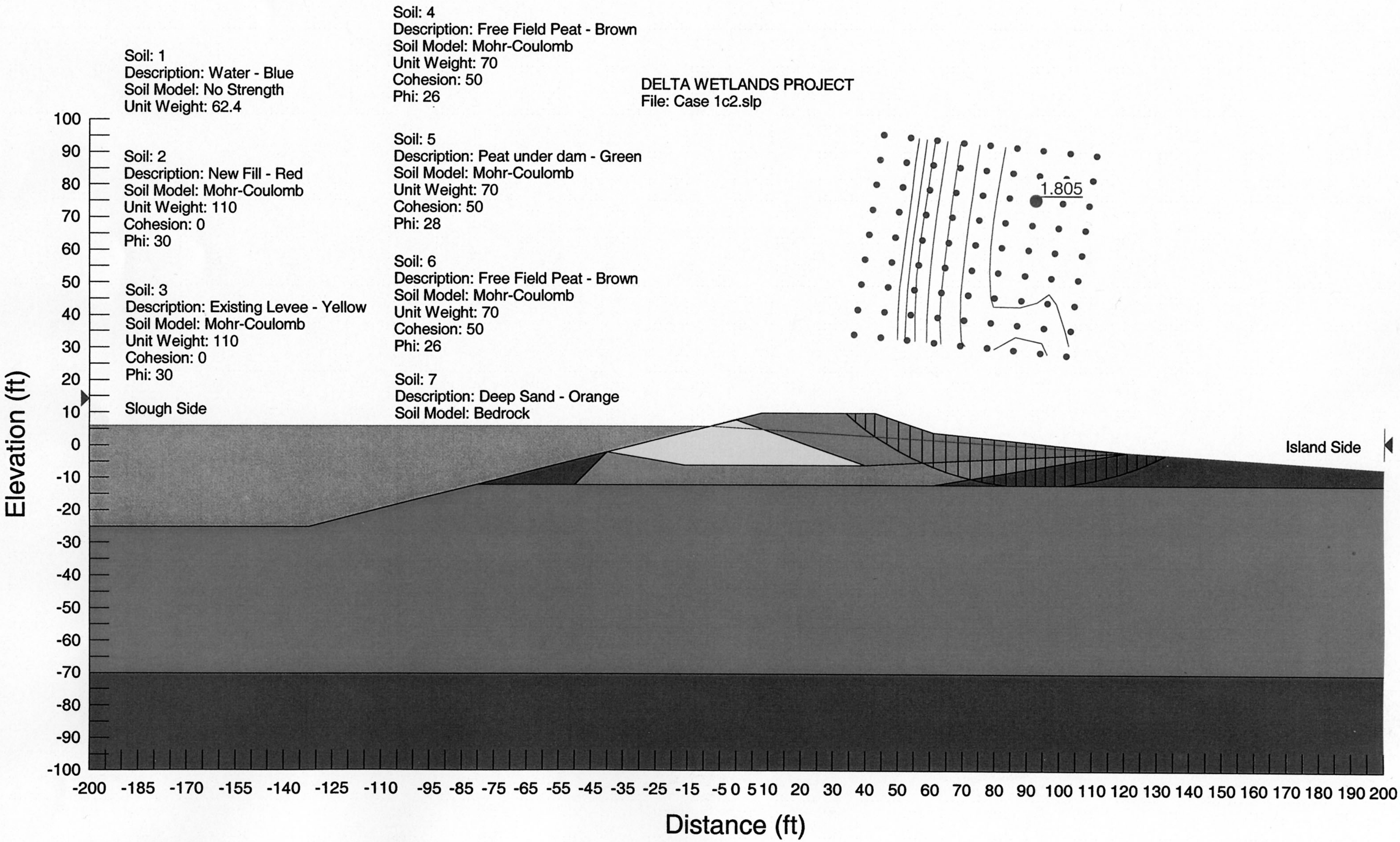
DELTA WETLANDS PROJECT
File: Case 6d.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
1.590
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250
Distance (ft)

c. Steady-state Condition with Sliding Towards the Island Side. - The analyses performed to date for levees and for a storage reservoir indicate that slopes on the island side need to be 5:1 or flatter or be 3:1 with a buttressing berm. A continuous slope such as 5:1 general requires a greater volume of material so a steeper slope with a buttressing berm is generally more economical as illustrated on figure x. Actual slopes should be based upon economics (quantity of fill required), staged construction requirements, and achieving a factor of safety of approximately 1.5. Based on previous analyses and some additional analyses, as shown on Table x, DWR/BOR recommends that at this level of study a slope of 3:1 down to elevation 4 and a slope of 10:1 below that elevation should be used. More complete analysis should be done during final design to optimize slopes for different reaches of the embankments with different geometry and foundation conditions.

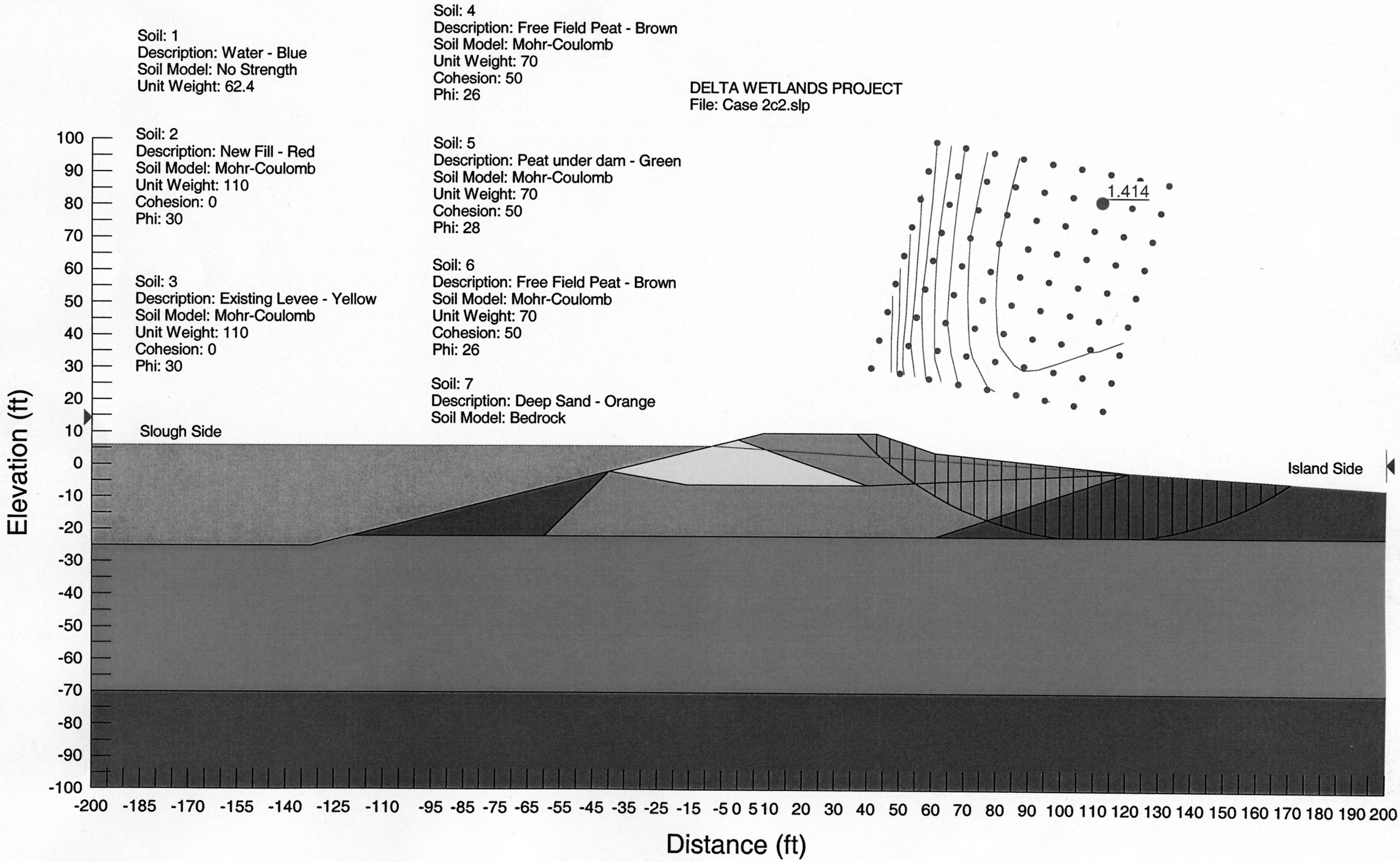
Table x. DWR/BOR Factors of Safety for Steady-state Condition and Sliding Towards Island

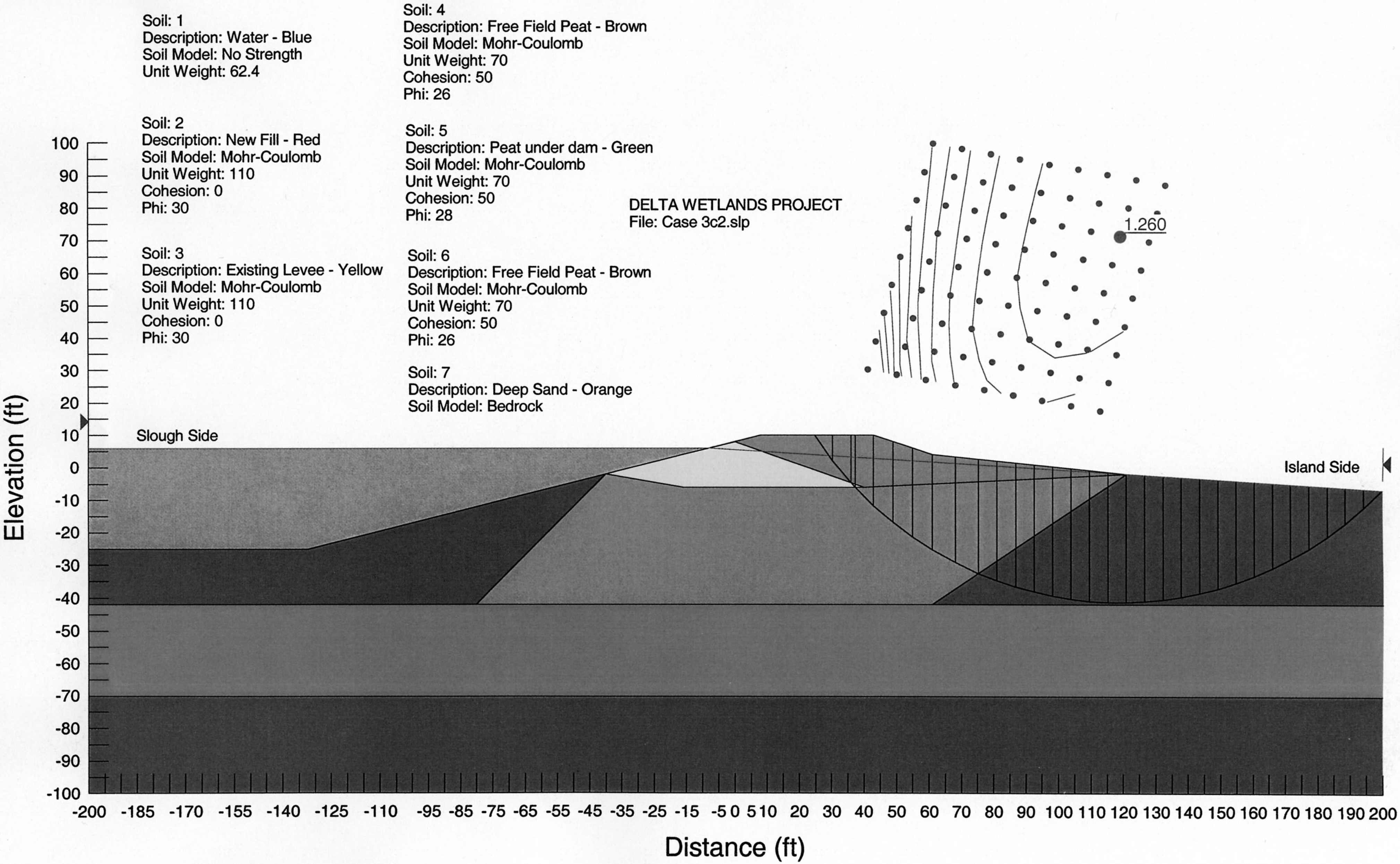
	1	2	3	4	5	6	7	8
Height of Existing Embankment, feet	10	10	10	24	24	24	16	16
Thickness of peat, feet	10	20	40	10	20	40	20	30
New Crest Elevation	10	10	10	10	10	10	15	15
Factor of Safety	1.80	1.41	1.26	2.71	1.96	1.49	1.67	1.46

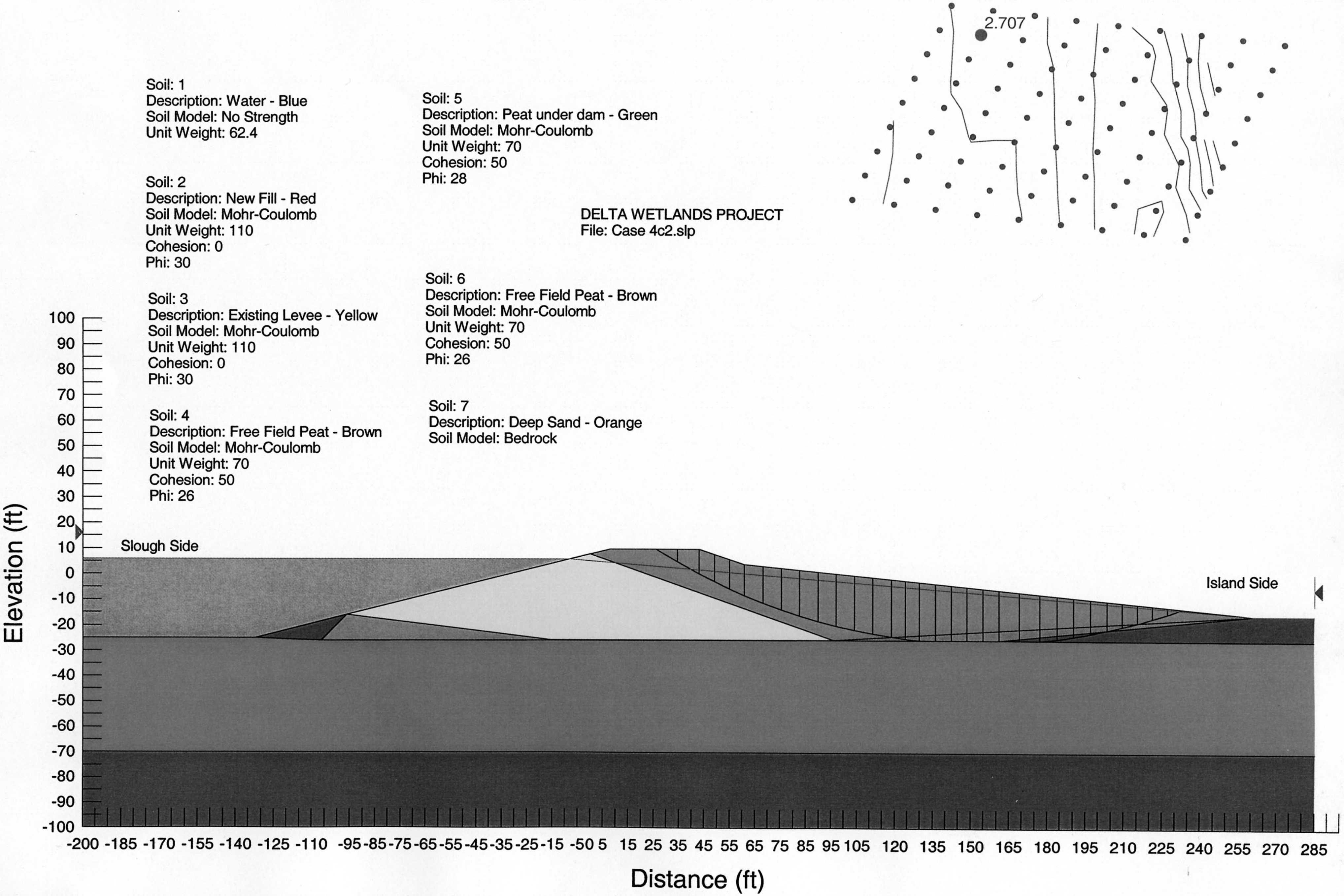
Assumes existing slope is approximately 4:1, new slope is 3:1 to elevation 4 and then 10:1, slough side slope is cut back to 4:1, and a new crest width of 35 feet, reservoir empty and river at elevation 6

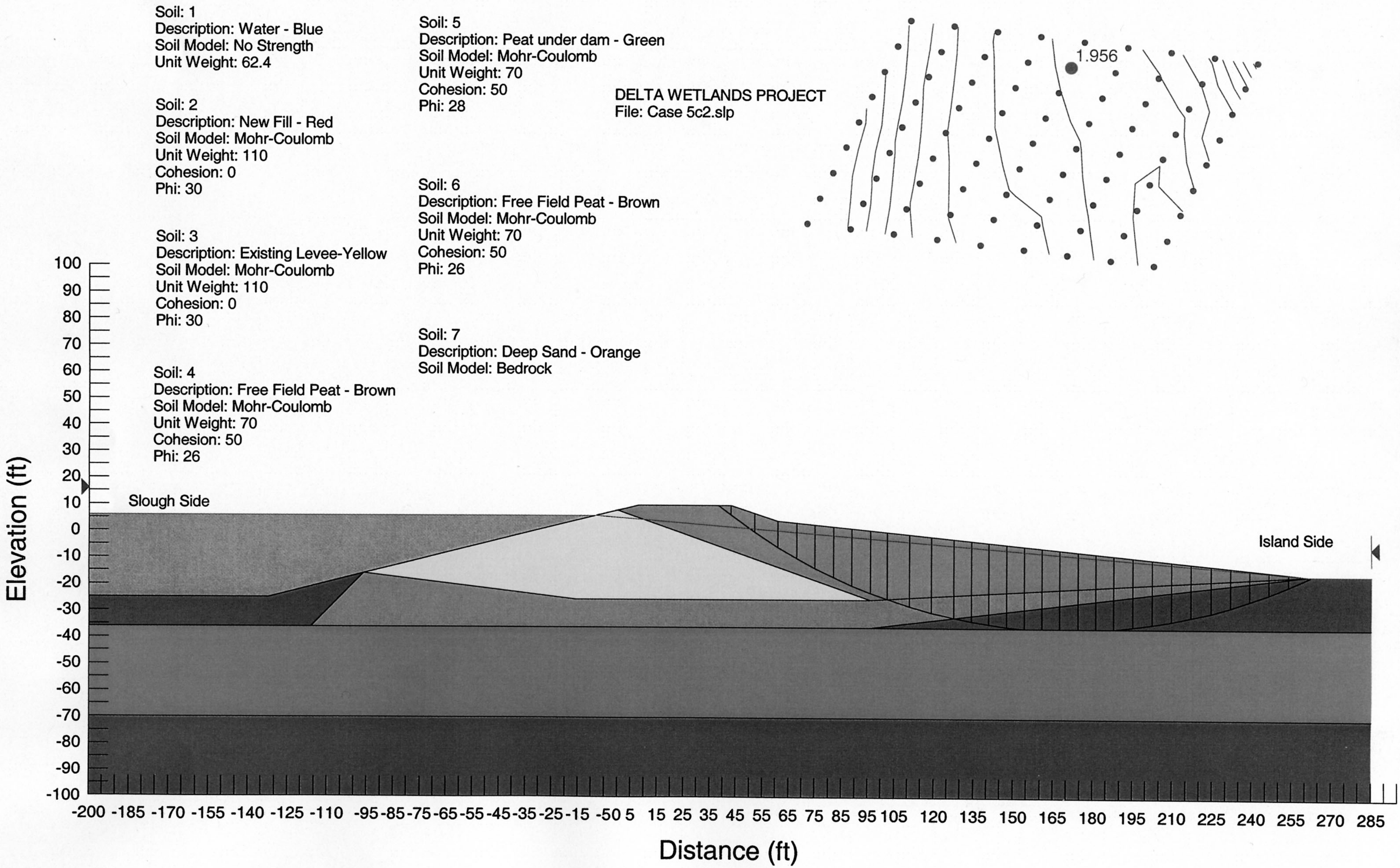


Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Slough Side
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 1c2.slp
1.805
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Distance (ft)

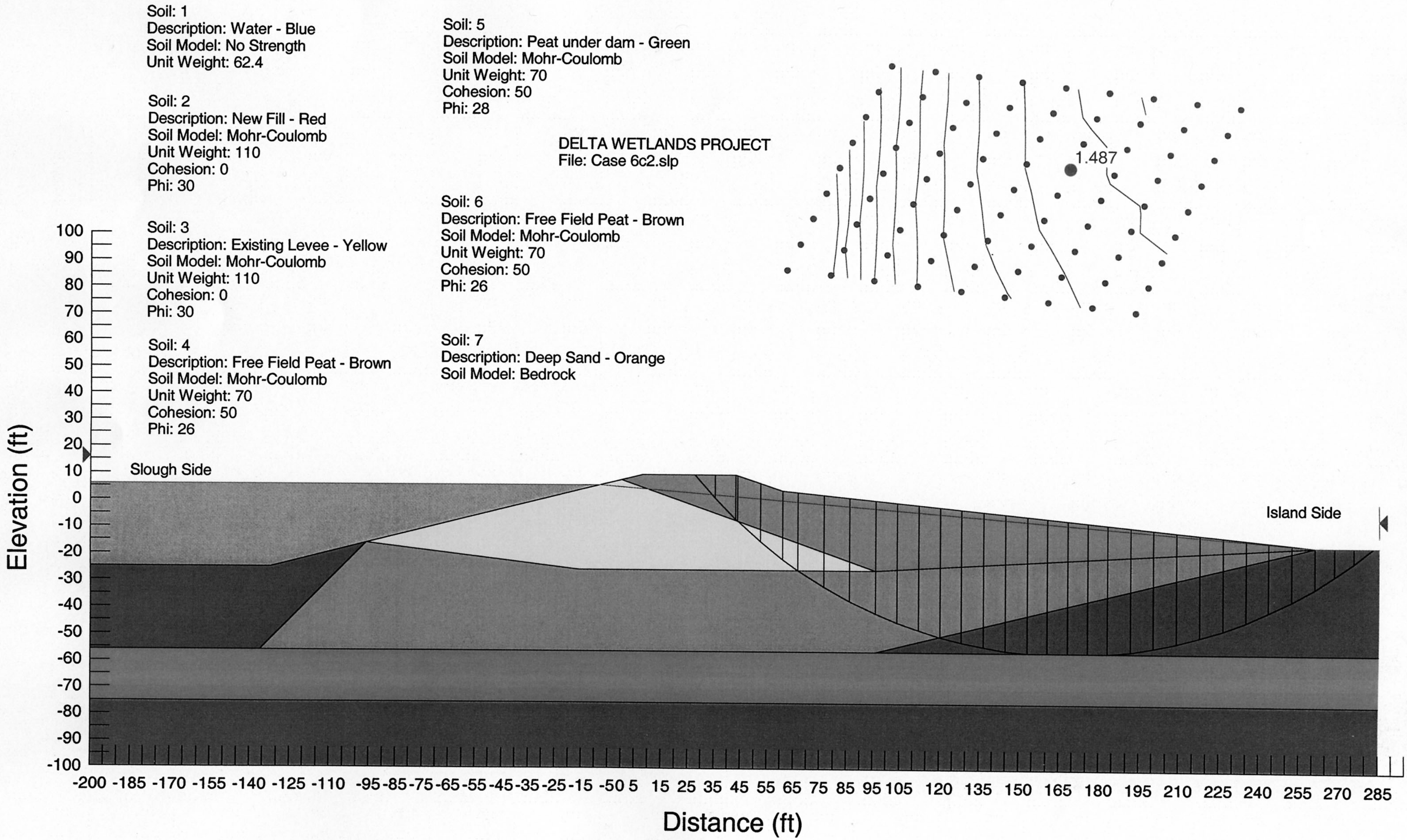

Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 2c2.slp
1.414
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Distance (ft)


Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 3c2.slp
1.260
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Distance (ft)

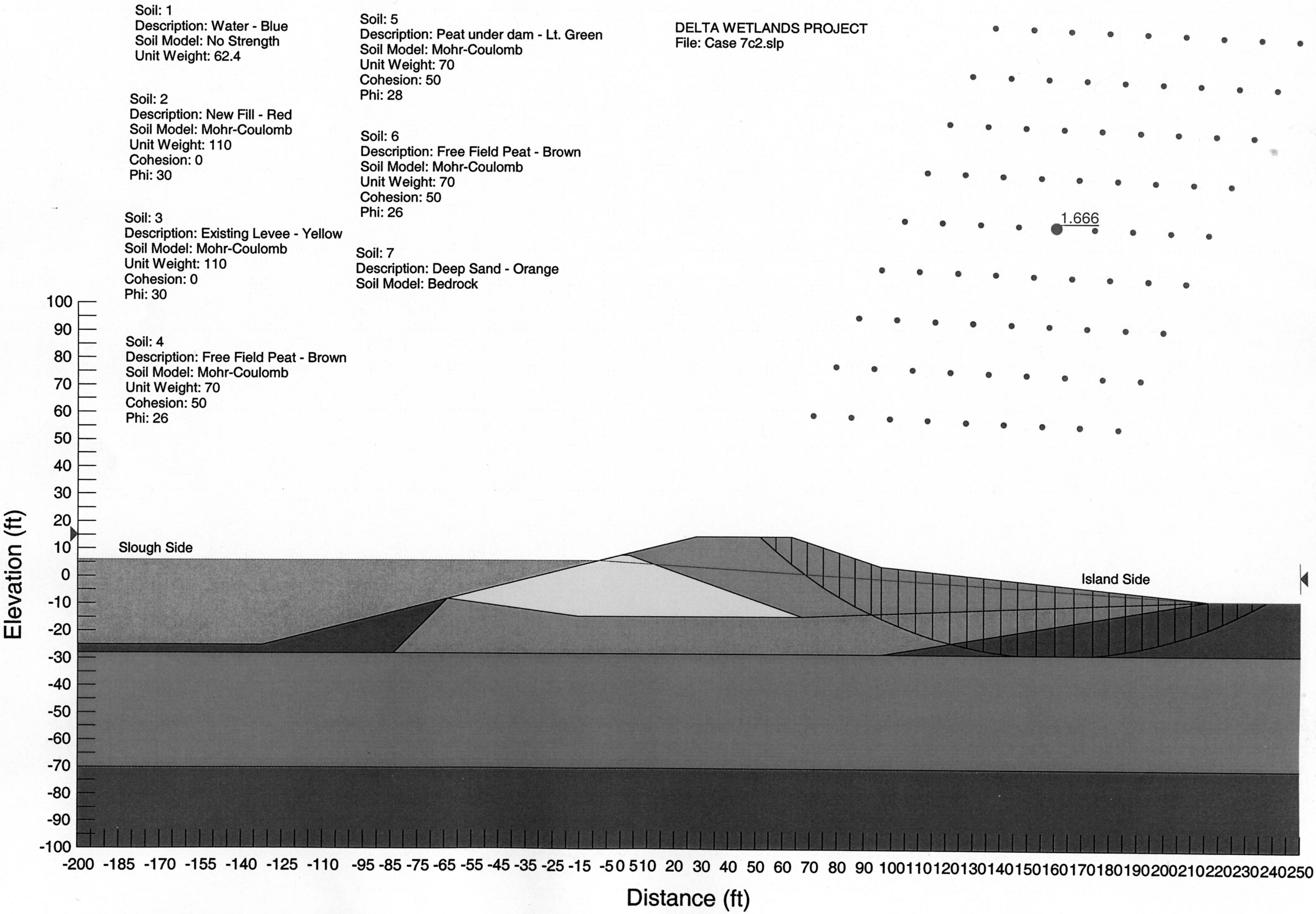

Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 4c2.slp
2.707
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 15 25 35 45 55 65 75 85 95 105 120 135 150 165 180 195 210 225 240 255 270 285
Distance (ft)

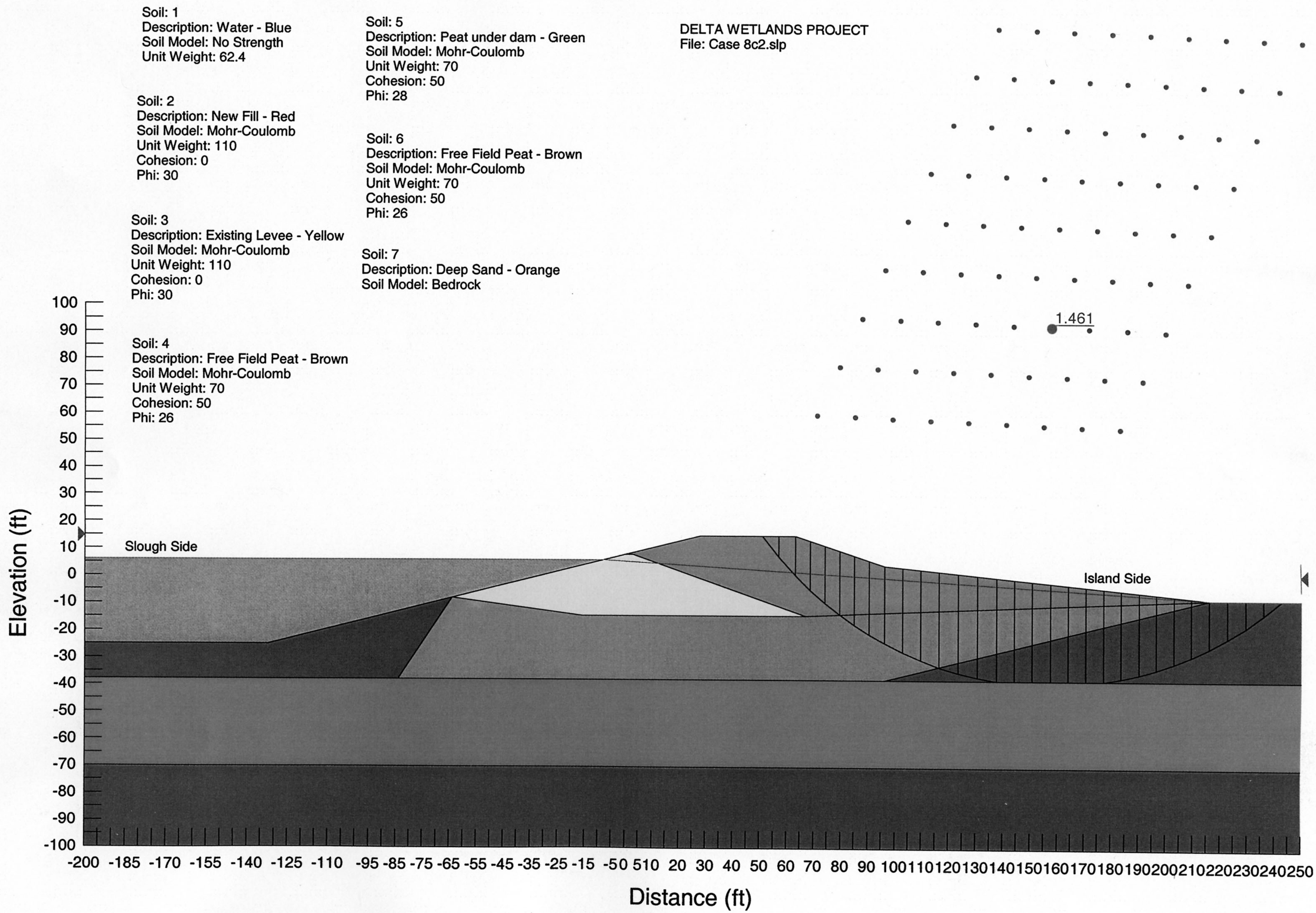


Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee-Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 5c2.slp
1.956
Slough Side
Island Side
Elevation (ft)
100 90 80 70 60 50 40 30 20 10 0 -10 -20 -30 -40 -50 -60 -70 -80 -90 -100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 15 25 35 45 55 65 75 85 95 105 120 135 150 165 180 195 210 225 240 255 270 285
Distance (ft)



Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
DELTA WETLANDS PROJECT
File: Case 6c2.slp
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
1.487
Slough Side
Island Side
Elevation (ft)
100 90 80 70 60 50 40 30 20 10 0 -10 -20 -30 -40 -50 -60 -70 -80 -90 -100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 15 25 35 45 55 65 75 85 95 105 120 135 150 165 180 195 210 225 240 255 270 285
Distance (ft)


Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Lt. Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 7c2.slp
1.666
Slough Side
Island Side
Elevation (ft)
Distance (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250



Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 8c2.slp
1.461
Slough Side
Island Side
Elevation (ft)
Distance (ft)

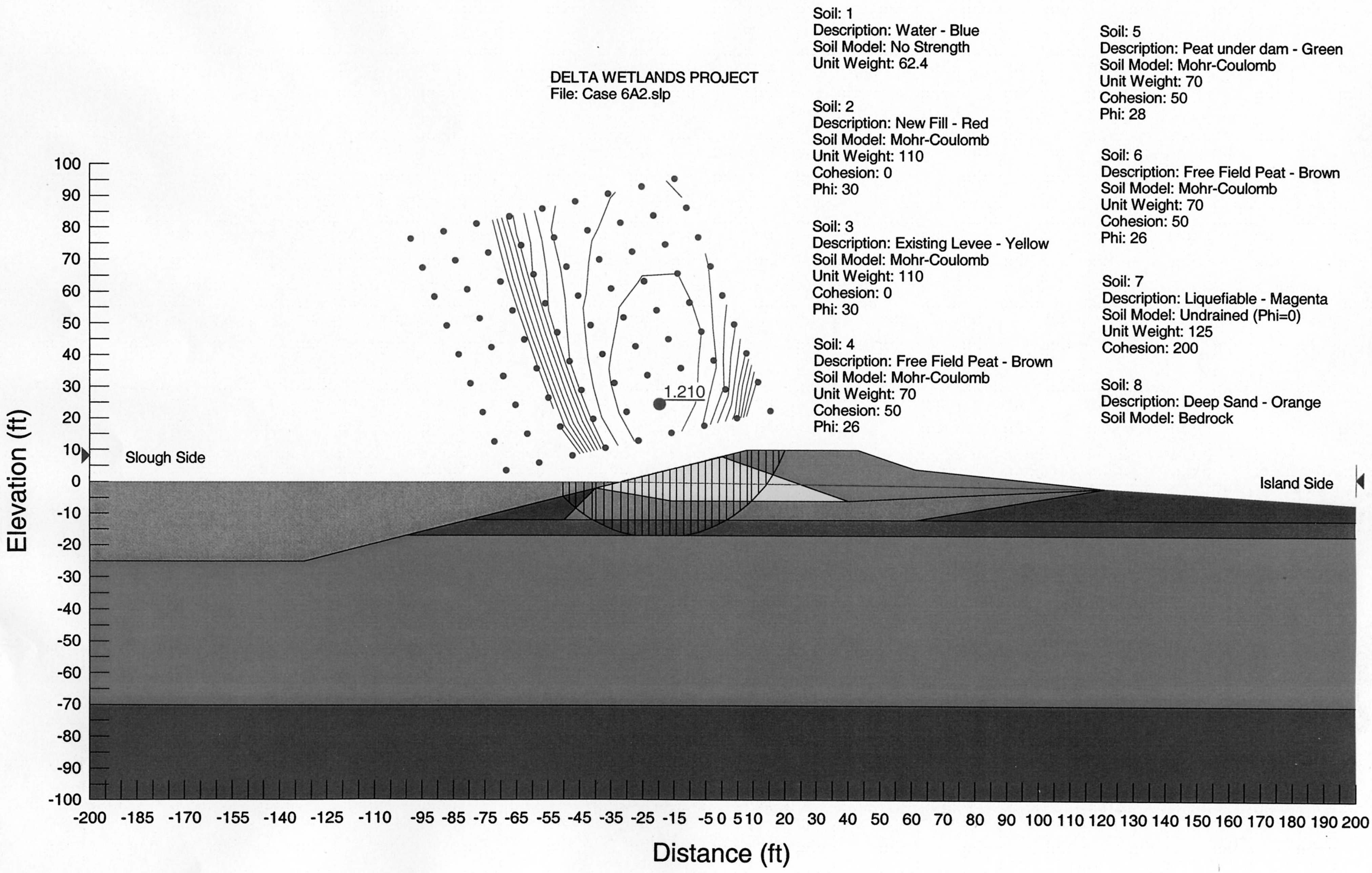
4.3.5 Post-liquefaction Stability Analysis

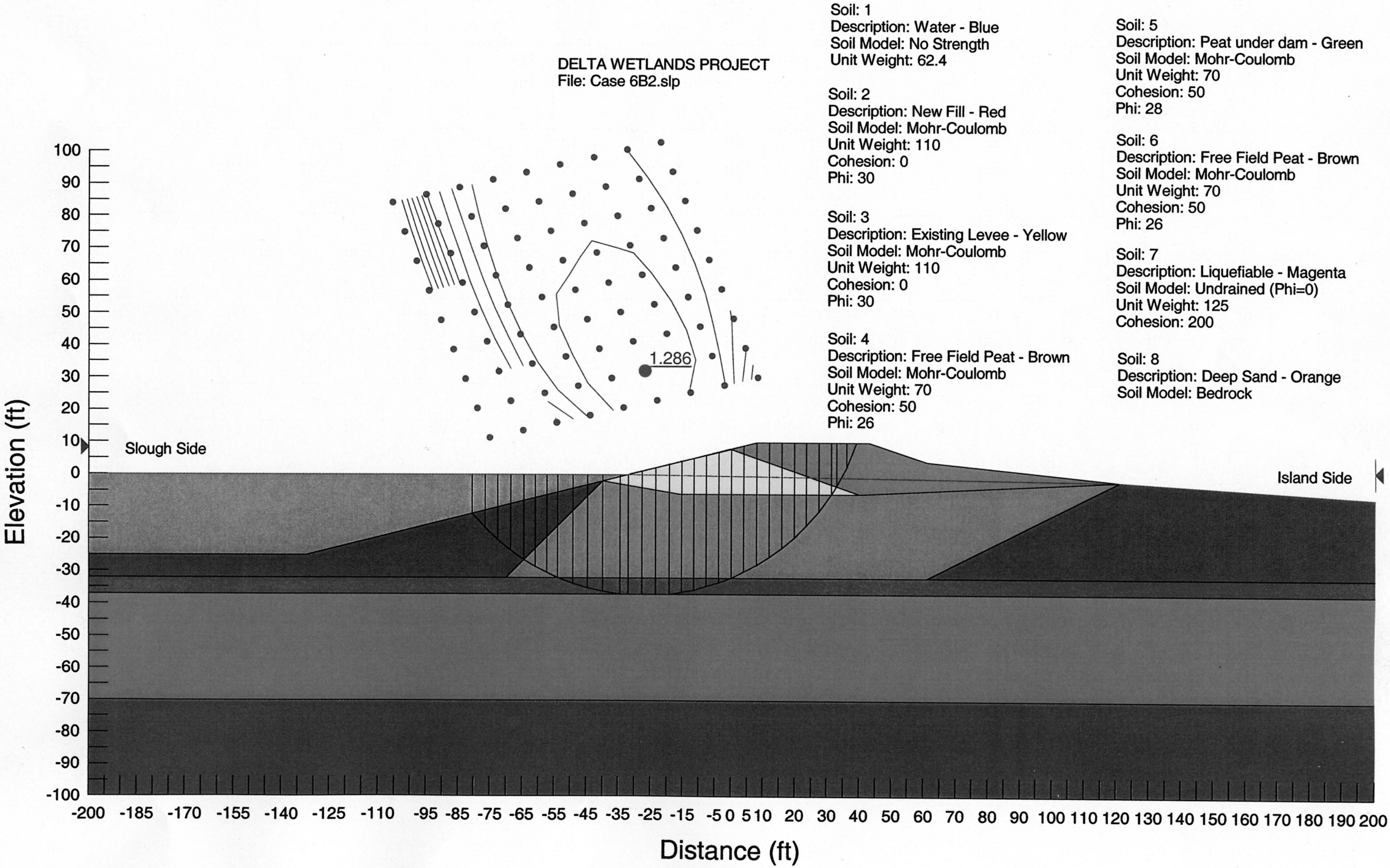
A post liquefaction analysis was performed on the same sections used for the steady-state analysis. Liquefied strengths of a small layer of material at the top of the sand layer was assumed to be 100, 200 and 400 psf. No other strength reductions were assumed. As shown in table x for sliding towards the river/slough if the liquefied strength is at least 200 psf a post-liquefaction sliding failure will not occur. As shown in table y for sliding towards the island if the liquefied strength is at least 200 psf a post-liquefaction sliding failure will probably not occur. During final design an in-depth analysis of the SPT data should be done to verify that the minimum liquefied strength is 200 psf. Future analyses should also evaluate the potential loss of strength in the peat material due to straining beyond the fiber bond strength.

Table x. DWR/BOR Factors of Safety for Post Liquefaction Condition and Sliding Towards River/Slough

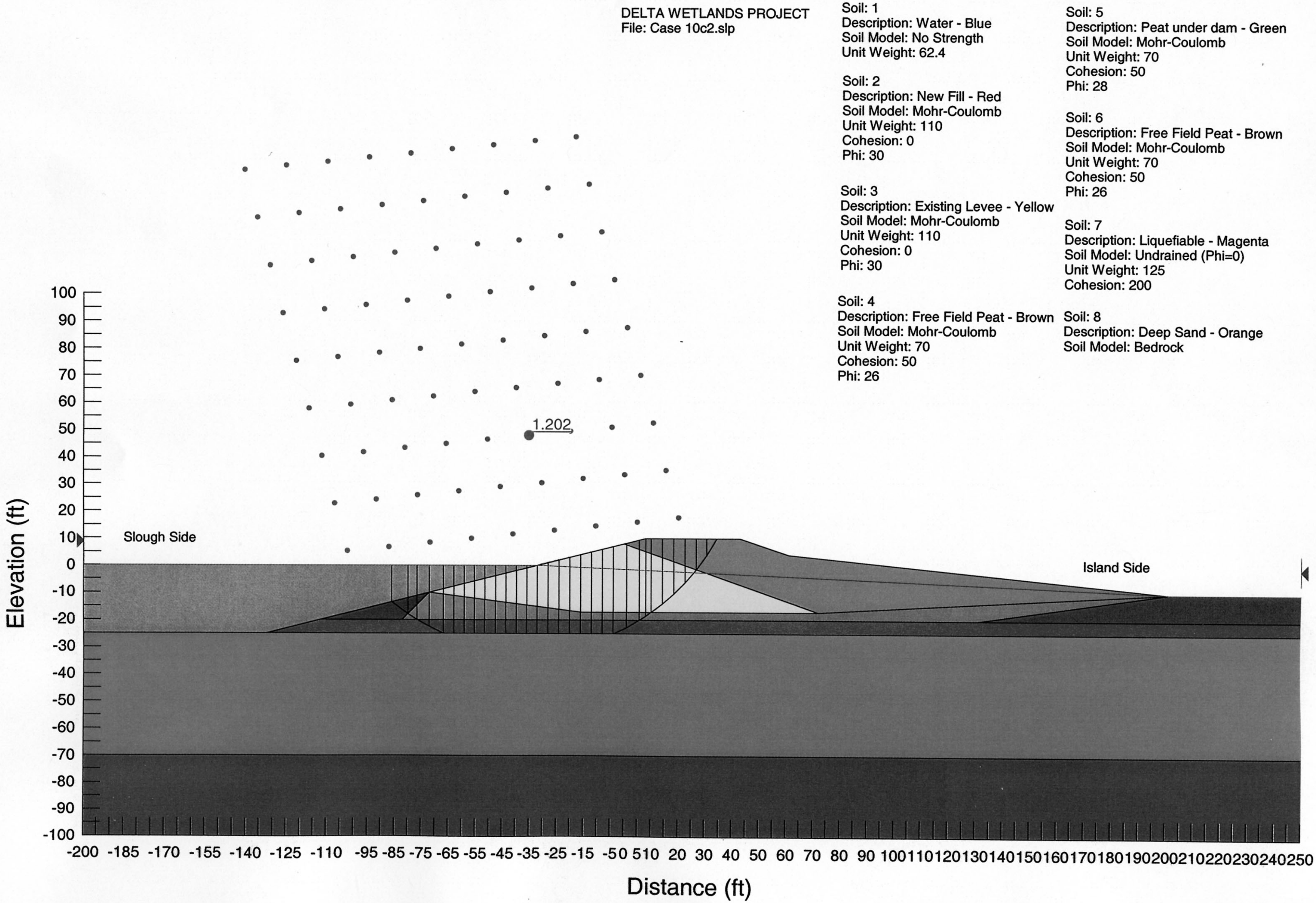
Liquefied Strength, psf	**Factor of Safety* 10' embankment**		**Factor of Safety* 18' embankment**	
	10' peat	30' peat	10' peat	30' peat
100	0.93	1.11	0.91	1.04
200	1.21	1.29	1.20	1.22
400	1.58	1.40	1.70	1.43
no liquef.	1.56	1.39	1.73	1.43

* Assumed 4:1 slope on the river/slough side, water in the slough to elevation 0 , no water in the reservoir, free field peat strength assumed to be c=50 psf and $\varphi = 26$, peat under embankment strength assumed to be c=50 psf and
$\varphi = 28$


DELTA WETLANDS PROJECT
File: Case 6A2.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
1.210
Slough Side
Island Side
Elevation (ft)
Distance (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200



DELTA WETLANDS PROJECT
File: Case 6B2.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
1.286
Slough Side
Island Side
Elevation (ft)
Distance (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200


DELTA WETLANDS PROJECT
File: Case 10c2.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
1.202
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250
Distance (ft)

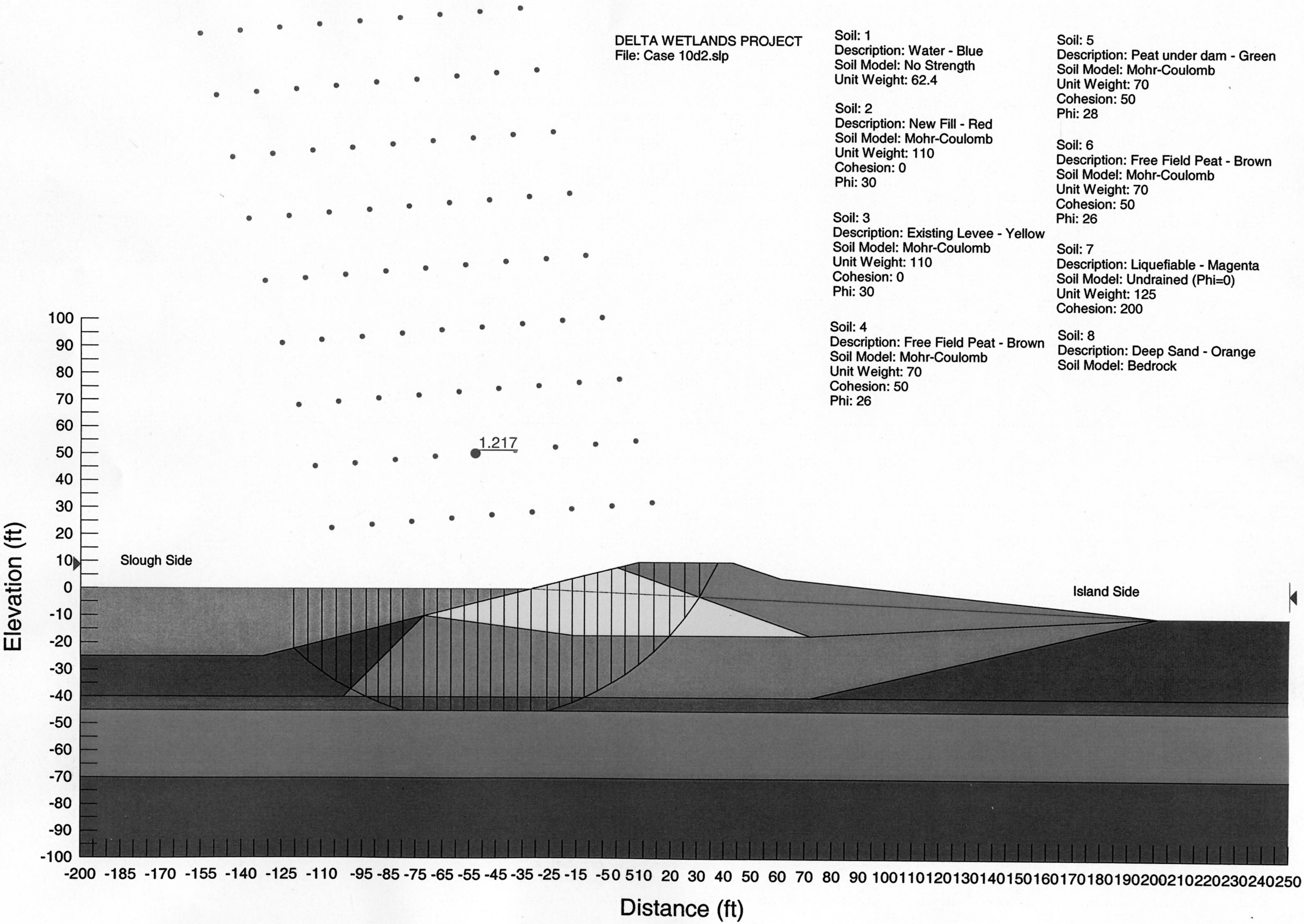
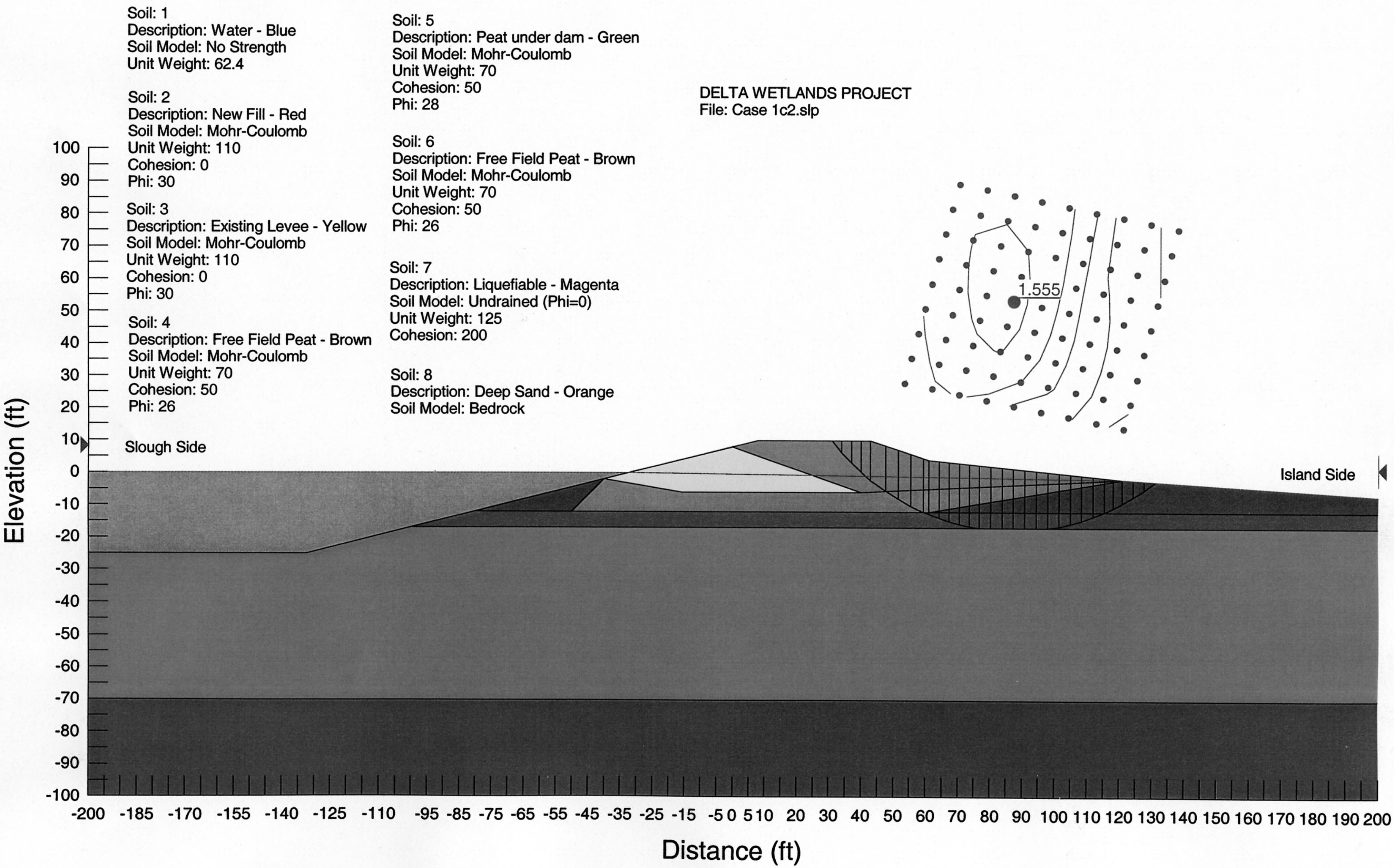

DELTA WETLANDS PROJECT
File: Case 10d2.slp
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
1.217
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250
Distance (ft)

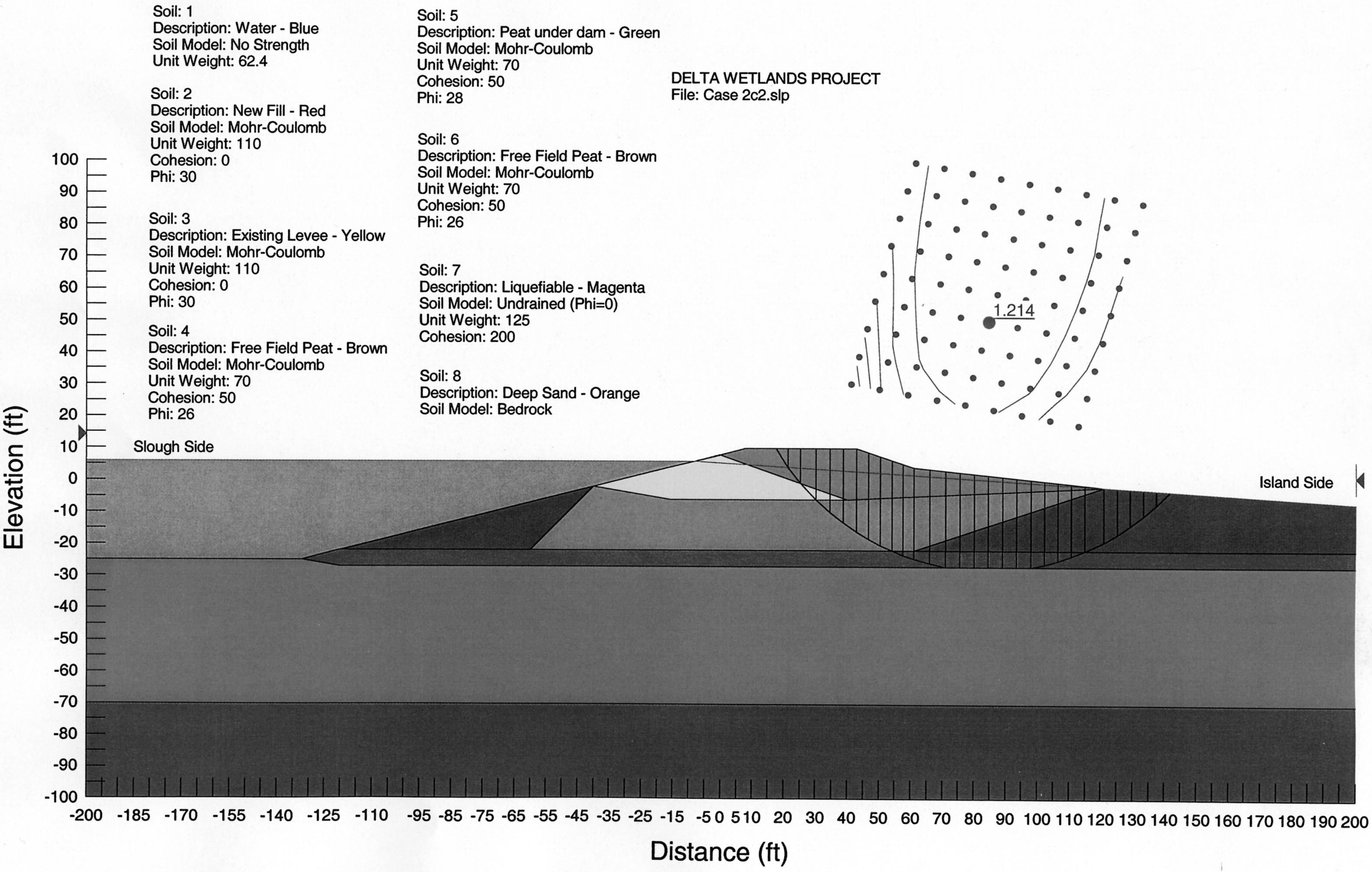
Table y. DWR/BOR Factors of Safety for Post Liquefaction Condition and Sliding Towards Island

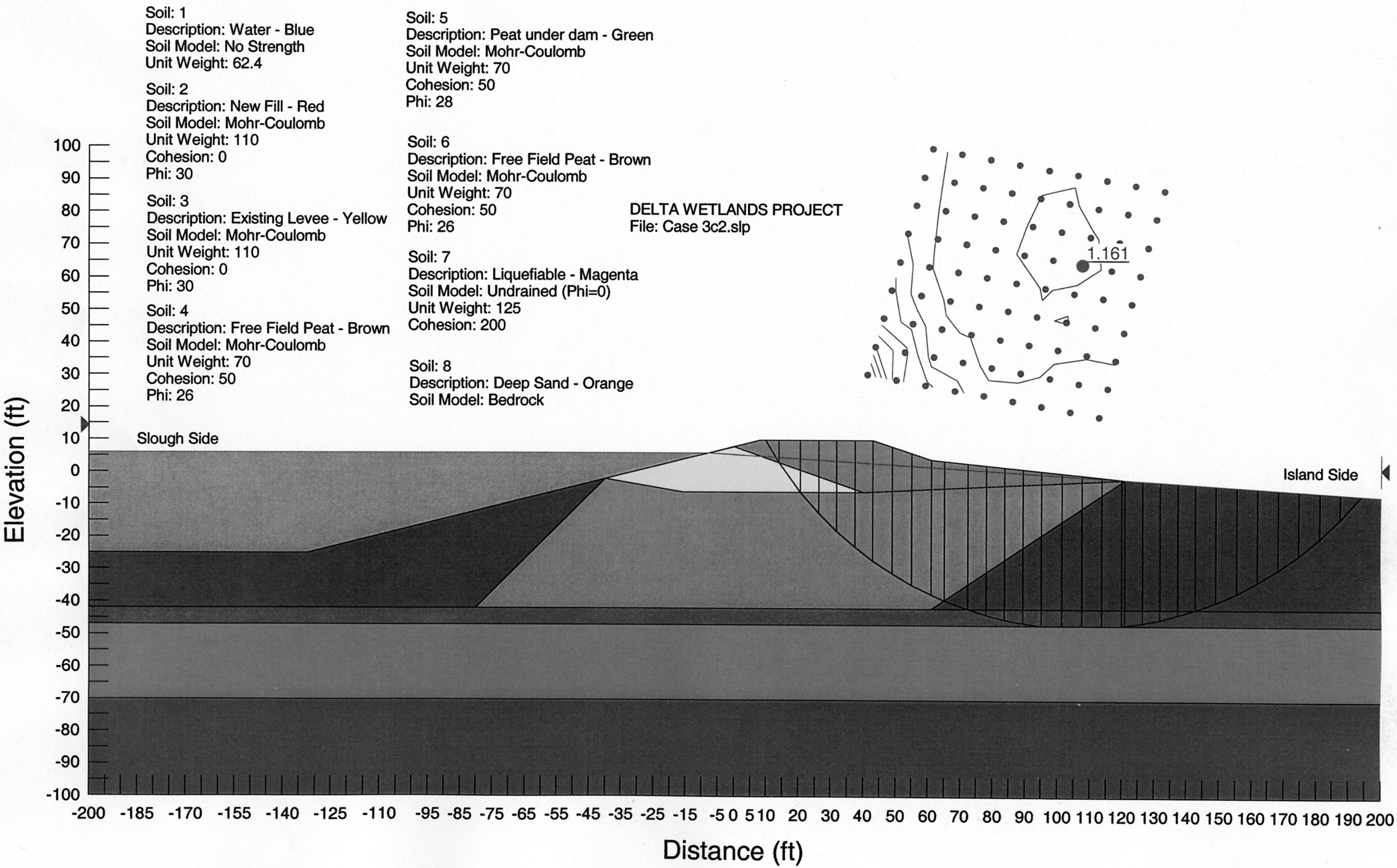
	1	2	3	4	5	6	7	8
Height of Existing Embankment, feet	10	10	10	24	24	24	16	16
Thickness of peat, feet	10	20	40	10	20	40	20	30
New Crest Elevation	10	10	10	10	10	10	15	15
Factor of Safety- no liquef.	1.80	1.41	1.26	2.71	1.96	1.49	1.67	1.46
Factor of Safety for 100 psf	1.21	1.0	1.02	1.23	1.06	0.98	0.98	0.92
Factor of Safety for 200 psf	1.55	1.21	1.16	1.49	1.24	1.07	1.17	1.06
Factor of Safety for 400 psf	2.16	1.43	1.26	1.97	1.56	1.26	1.49	1.32

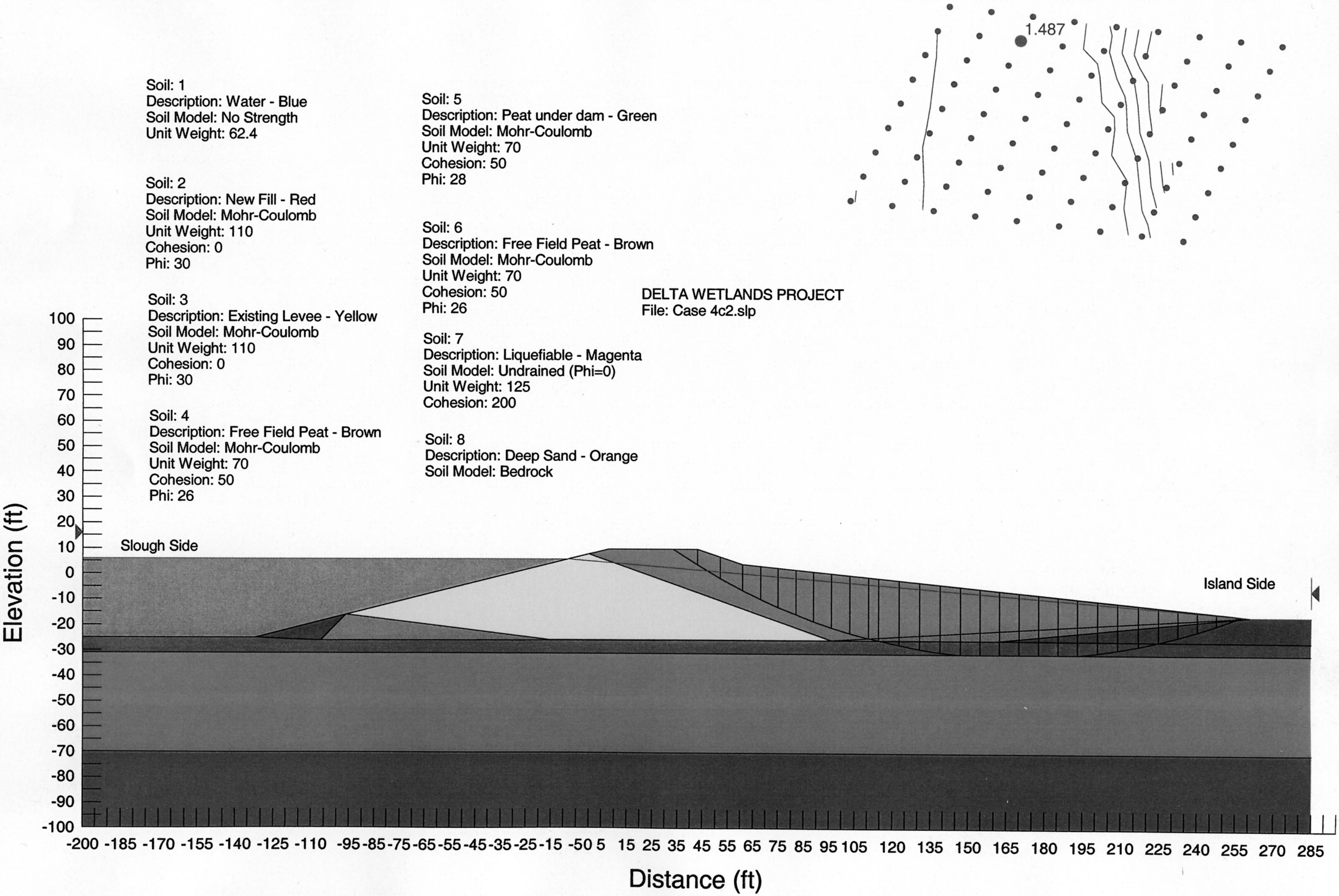
Assumes existing slope is approximately 4:1, new slope is 3:1 to elevation 4 and then 10:1, slough side slope is cut back to 4:1, and a new crest width of 35 feet, reservoir empty and river at elevation 6



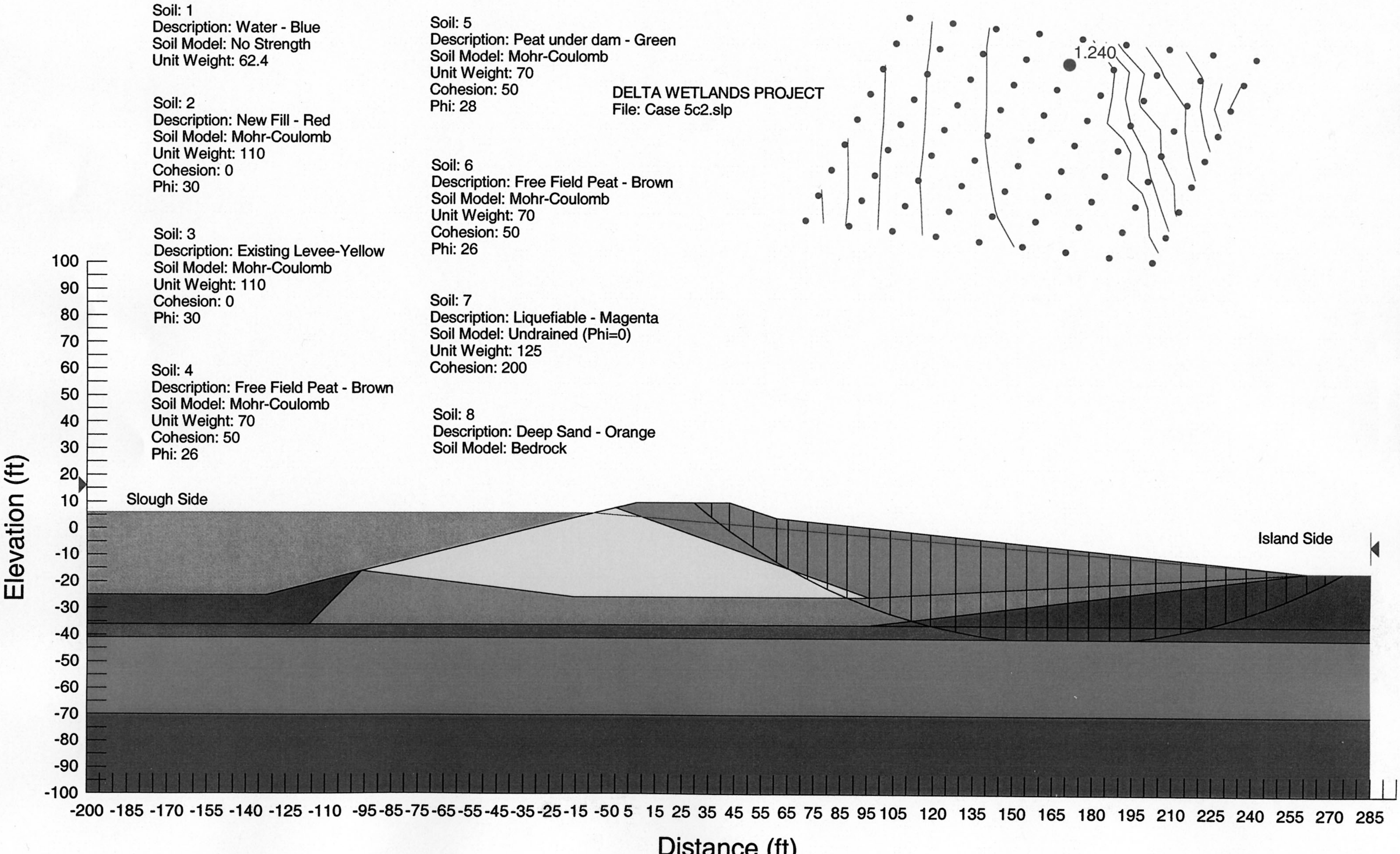
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 1c2.slp
1.555
Slough Side
Island Side
Elevation (ft)
Distance (ft)

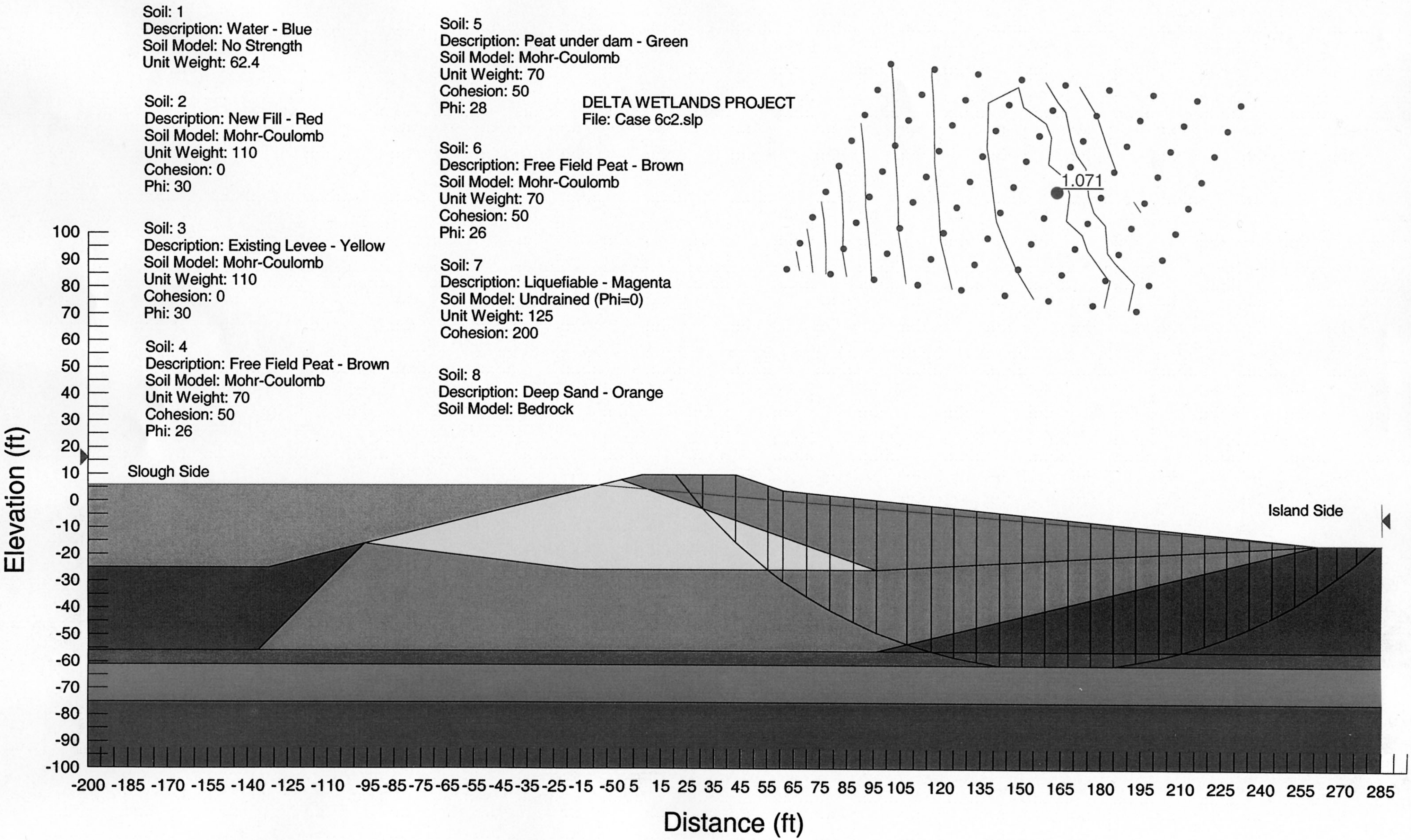

Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 2c2.slp
1.214
Slough Side
Island Side
Elevation (ft)
Distance (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200

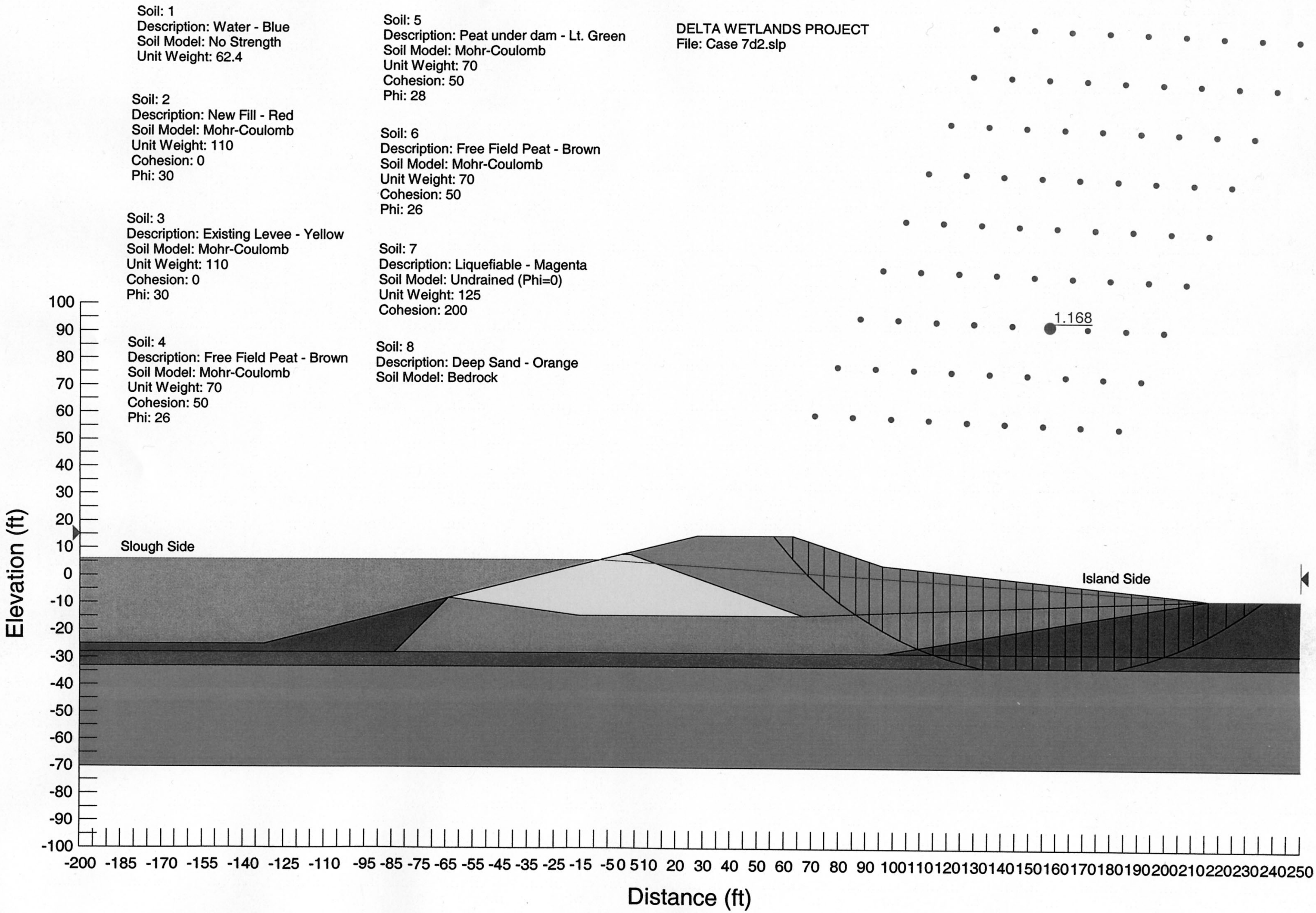

Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 3c2.slp
1.161
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
Distance (ft)

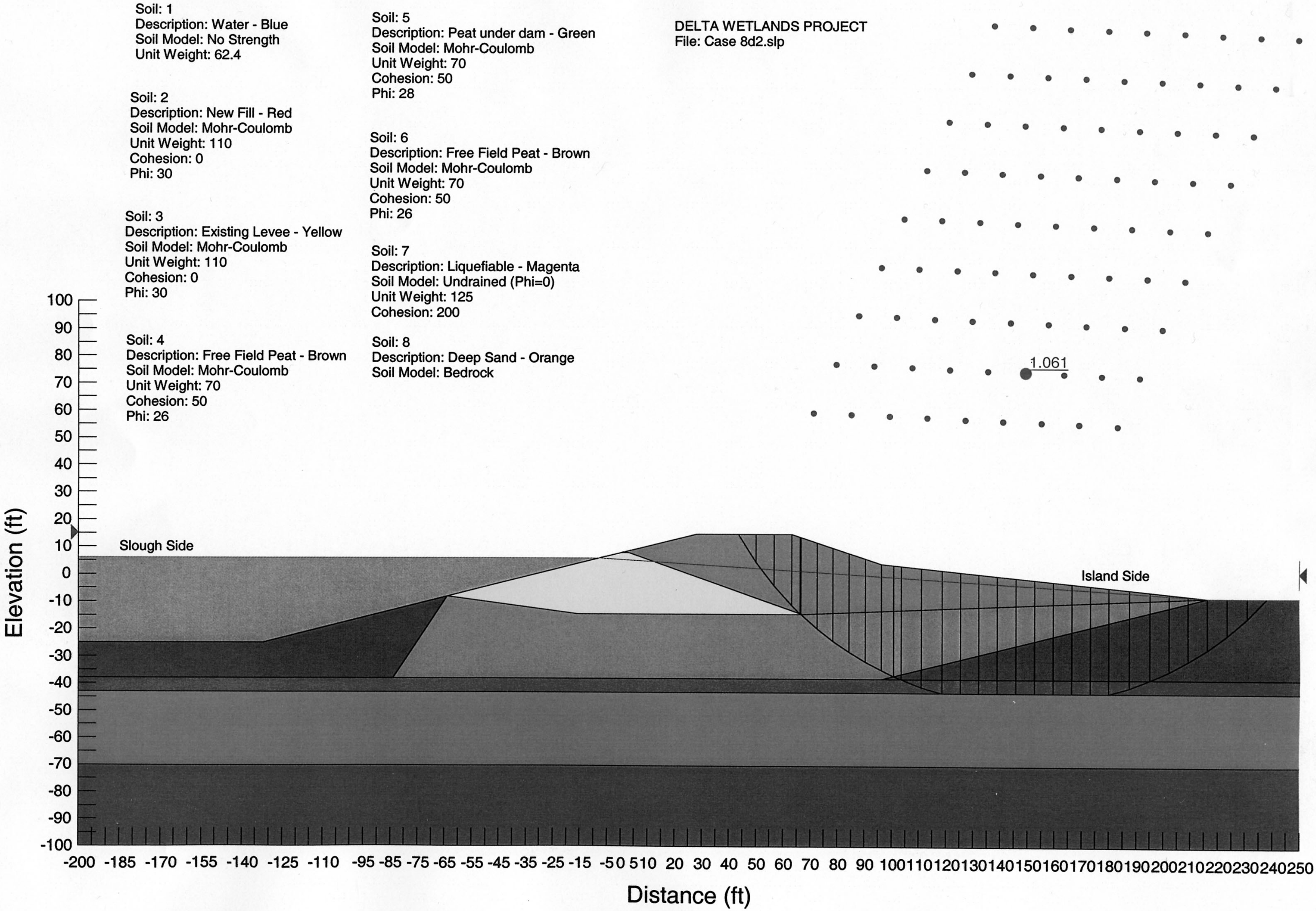


1.487
Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 4c2.slp
Slough Side
Island Side
Elevation (ft)
100 90 80 70 60 50 40 30 20 10 0 -10 -20 -30 -40 -50 -60 -70 -80 -90 -100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -50 5 15 25 35 45 55 65 75 85 95 105 120 135 150 165 180 195 210 225 240 255 270 285
Distance (ft)


Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee-Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 5c2.slp
1.240
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 15 25 35 45 55 65 75 85 95 105 120 135 150 165 180 195 210 225 240 255 270 285
Distance (ft)


Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 6c2.slp
1.071
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 5 15 25 35 45 55 65 75 85 95 105 120 135 150 165 180 195 210 225 240 255 270 285
Distance (ft)


Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Lt. Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 7d2.slp
1.168
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250
Distance (ft)



Soil: 1
Description: Water - Blue
Soil Model: No Strength
Unit Weight: 62.4
Soil: 2
Description: New Fill - Red
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 3
Description: Existing Levee - Yellow
Soil Model: Mohr-Coulomb
Unit Weight: 110
Cohesion: 0
Phi: 30
Soil: 4
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 5
Description: Peat under dam - Green
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 28
Soil: 6
Description: Free Field Peat - Brown
Soil Model: Mohr-Coulomb
Unit Weight: 70
Cohesion: 50
Phi: 26
Soil: 7
Description: Liquefiable - Magenta
Soil Model: Undrained (Phi=0)
Unit Weight: 125
Cohesion: 200
Soil: 8
Description: Deep Sand - Orange
Soil Model: Bedrock
DELTA WETLANDS PROJECT
File: Case 8d2.slp
1.061
Slough Side
Island Side
Elevation (ft)
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
-200 -185 -170 -155 -140 -125 -110 -95 -85 -75 -65 -55 -45 -35 -25 -15 -5 0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 210 220 230 240 250
Distance (ft)